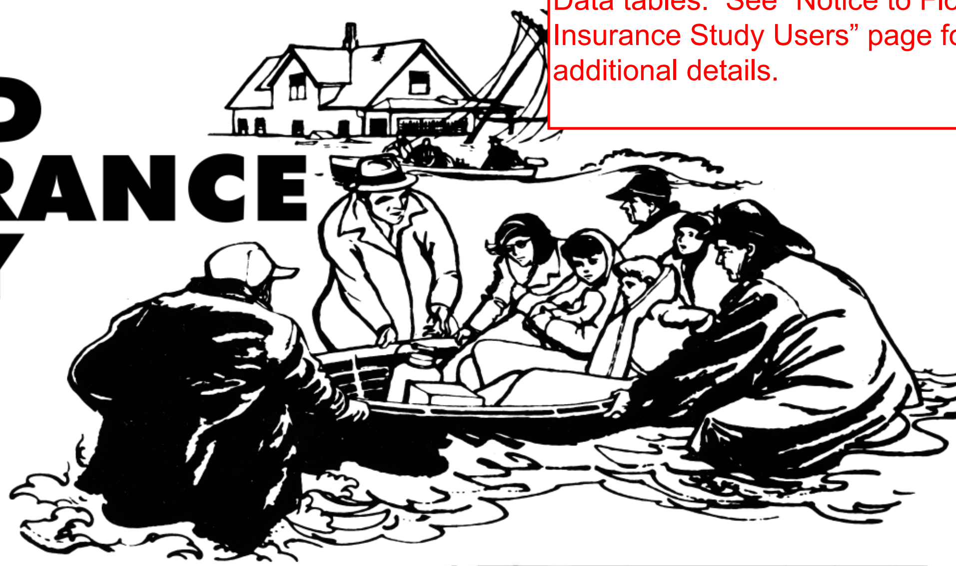
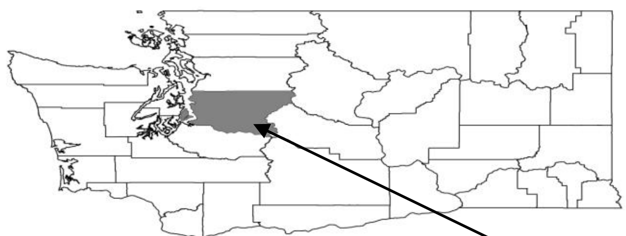
Notice

This preliminary FIS report includes only revised Flood Profiles and Floodway Data tables. See “Notice to Flood Insurance Study Users” page for additional details.

FLOOD INSURANCE STUDY



KING COUNTY, WASHINGTON AND INCORPORATED AREAS



Volume 1 of 4

COMMUNITY NAME	COMMUNITY NUMBER
*ALGONA, CITY OF	530072
AUBURN, CITY OF	530073
*BEAUX ARTS VILLAGE, TOWN OF	530242
BELLEVUE, CITY OF	530074
BLACK DIAMOND, CITY OF	530272
BOTHELL, CITY OF	530075
BURIEN, CITY OF	530321
CARNATION, CITY OF	530076
*CLYDE HILL, CITY OF	530279
COVINGTON, CITY OF	530339
DES MOINES, CITY OF	530077
DUVALL, CITY OF	530282
ENUMCLAW, CITY OF	530319
FEDERAL WAY, CITY OF	530322
*HUNTS POINT, TOWN OF	530288
ISSAQUAH, CITY OF	530079
KENMORE, CITY OF	530336
KENT, CITY OF	530080
KING COUNTY, UNINCORPORATED AREAS	530071
KIRKLAND, CITY OF	530081
LAKE FOREST PARK, CITY OF	530082
*MAPLE VALLEY, CITY OF	530078
*MEDINA, CITY OF	530315
*MERCER ISLAND, CITY OF	530083
MUCKLESHOOT INDIAN RESERVATION	530165
NEWCASTLE, CITY OF	530134
NORMANDY PARK, CITY OF	530084
NORTH BEND, CITY OF	530085
PACIFIC, CITY OF	530086
REDMOND, CITY OF	530087
RENTON, CITY OF	530088
SAMMAMISH, CITY OF	530337
SEATAC, CITY OF	590320
SEATTLE, CITY OF	530089
SHORELINE, CITY OF	530327
SKYKOMISH, TOWN OF	530236
SNOQUALMIE, CITY OF	530090
TUKWILA, CITY OF	530091
WOODINVILLE, CITY OF	530324
*YARROW POINT, TOWN OF	530309

*No Special Flood Hazard Areas Identified

PRELIMINARY:

Federal Emergency Management Agency

Flood Insurance Study Number
53033CV001B

NOTICE TO
FLOOD INSURANCE STUDY USERS

Communities participating in the National Flood Insurance Program have established repositories of flood hazard data for floodplain management and flood insurance purposes. This Flood Insurance Study (FIS) report may not contain all data available within the Community Map Repository. Please contact the Community Map Repository for any additional data.

The Federal Emergency Management Agency (FEMA) may revise and republish part or all of this FIS report at any time. In addition, FEMA may revise part of this FIS report by the Letter of Map Revision process, which does not involve republication or redistribution of the FIS report. Therefore, users should consult with community officials and check the Community Map Repository to obtain the most current FIS report components.

Selected Flood Insurance Rate Map panels for this community contain information that was previously shown separately on the corresponding Flood Boundary and Floodway Map panels (e.g., floodways, cross sections). In addition, former flood hazard zone designations have been changed as follows:

Old Zone(s)	New Zone
Al through A30	AE
V1 through V30	VE
B	X
C	X

Initial Countywide FIS Effective Date: September 29, 1989

Revised Countywide Date(s): May 16, 1995
May 20, 1996
March 30, 1998
November 8, 1999
December 6, 2001
April 19, 2005
To Be Determined

This preliminary FIS report does not include unrevised Floodway Data Tables or unrevised Flood Profiles. These Floodway Data Tables and Flood Profiles will appear in the final FIS report.

TABLE OF CONTENTS

Volume 1

Page

Table of Contents (continued)

Volume 1

Table of Contents (continued)

Volume 1

Volume 2

Table of Contents (continued)

FIGURES

Volume 2

TABLES

Volume 1

Volume 2

EXHIBIT

Volume 2

Table of Contents (continued)

EXHIBIT (continued)

Volume 2 (continued)

Exhibit 1 – Flood Profiles (continued)

Volume 3

Volume 4

Table of Contents (continued)

EXHIBIT (continued)

Volume 4 (continued)

Exhibit 1 – Flood Profiles (continued)

PUBLISHED SEPARATELY

FLOOD INSURANCE STUDY
KING COUNTY, WASHINGTON AND INCORPORATED AREAS

1.0 INTRODUCTION

1.1 Purpose of Study

This Flood Insurance Study (FIS) investigates the existence and severity of flood hazards in the geographic area of King County, Washington, including the Cities of Algona, Auburn, Bellevue, Black Diamond, Bothell, Burien, Carnation, Clyde Hill, Covington, Des Moines, Duvall, Enumclaw, Federal Way, Issaquah, Kenmore, Kent, Kirkland, Lake Forest Park, Maple Valley, Medina, Mercer Island, Newcastle, Normandy Park, North Bend, Pacific, Redmond, Renton, Sammamish, SeaTac, Seattle, Shoreline, Snoqualmie, Tukwila, Woodinville, the Towns of Beaux Arts Village, Hunts Point, Skykomish, Yarrow Point, the Muckleshoot Indian Reservation, and the unincorporated areas of King County (hereinafter referred to collectively as King County), and aids in the administration of the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973. This study has developed flood risk data for various areas of the community that will be used to establish actuarial flood insurance rates and to assist the community in its efforts to promote sound floodplain management. Minimum floodplain management requirements for participation in the National Flood Insurance Program (NFIP) are set forth in the Code of Federal Regulations at 44 CFR, 60.3.

Please note that the City of Milton is geographically located in King and Pierce Counties. The flood-hazard information for the City of Milton is for information purposes only. See Pierce County separately published FIS report and FIRM for City of Milton.

Please note that the Cities of Algona, Clyde Hill, Maple Valley, Medina, and Mercer Island and the Towns of Beaux Arts Village, Hunts Point, and Yarrow Point have No Special Flood Hazard Areas Identified.

In some States or communities, floodplain management criteria or regulations may exist that are more restrictive or comprehensive than the minimum Federal requirement. In such cases, the more restrictive criteria take precedence and the State (or other jurisdictional agency) will be able to explain them.

1.2 Authority and Acknowledgments

The sources of authority for this FIS are the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973.

The hydrologic and hydraulic analyses for the original King County study were performed by the U.S. Army Corps of Engineers (USACE), Seattle District, for the Federal Emergency Management Agency (FEMA), under Inter-Agency Agreement No. IAA-H-2-73, Project Order No. 14, and Inter-Agency Agreement No. IAA-H-19-74, Project Order Nos. 1 and 15. This study was completed in August 1976. The Enplan Corporation, Consulting Engineers, Kirkland, Washington, assisted in the transfer of map data from photomosaic and topographic maps to the report work maps for the Seattle District, USACE.

The hydrologic and hydraulic analyses for the Tolt River were performed by the U.S. Soil Conservation Service (SCS) for Flood Hazard Analyses, Tolt River, and King County, Washington.

Hydrologic and hydraulic analyses for the communities of King County were performed by study contractors and are summarized below:

Community	Contractor	Contract Number	Completion Date
King County (revised study)	CH2M Hill Northwest, Inc., for FEMA	EMW-85-C-1893	June 1987
City of Seattle (revised study)	CH2M Hill Northwest, Inc., for FEMA	EMW-85-C-1893	June 1987
Portion of Upper Green River Valley upstream from Auburn	USACE, Seattle District, for FEMA	Inter-Agency Agreement No. IAA-EMW-E-1153 Project Order No. 1	February 1988
City of Auburn (original study)	Tudor Engineering Co., for FEMA	H-4025, Amendment 4	May 1978
City of Auburn (revised study)	CH2M Hill Northwest, Inc., for FEMA	EMW-85-C-1893	June 1987

Community	Contractor	Contract Number	Completion Date
City of Bellevue	USGS, Water Resources Division for FEMA	Inter-Agency Agreement No. IAA-H-8-76, Project Order No. 3	May 1977
City of Carnation	CH2M Hill, Inc., for FEMA	H-4600	August 1978
	Harper Houf Righeliis Inc.	N/A	May 2002
City of Des Moines	CH2M Hill, Inc., for FEMA	H-4600	September 1978
City of Duvall	CH2M Hill, Inc., for FEMA	H-4600	September 1978
	NHC Inc.	N/A	N/A
City of Issaquah	Tudor Engineering Co. for FEMA	H-4025	September 1977
City of Kent (original study)	Tudor Engineering Co. for FEMA	H-4025 Amendment No. 13	June 1979
City of Kent (revised study)	CH2M Hill Northwest, Inc., for FEMA	EMW-85-C-1893	June 1988
City of Kirkland	Tudor Engineering Co., for FEMA	H-4025	December 1977
City of Lake Forest Park	CH2M Hill, Inc., for FEMA	H-4600	August 1978
City of Normandy Park	CH2M Hill, Inc., for FEMA	H-3815	June 1976
City of North Bend	CH2M Hill, Inc., for FEMA	H-4600	October 2001
City of Pacific	CH2M Hill, Inc., for FEMA	H-4600	April 1979

Community	Contractor	Contract Number	Completion Date
City of Redmond	Tudor Engineering Co., for FEMA	N/A	August 1977
City of Redmond (additional hydrologic and hydraulic analyses)	USACE, Seattle District for FEMA	N/A	August 1976
City of Renton (original study)	Tudor Engineering Co. for FEMA	H-4025	July 1979
City of Renton (revised study)	CH2M Hill, Northwest, Inc., for FEMA	EMW-85-C-1893	June 1987
Town of Skykomish	CH2M Hill, Inc., for FEMA	H-4600	July 1979
Town of Snoqualmie	CH2M Hill, Inc., for FEMA (additional data from USACE)	H-4810	July 1981
City of Tukwila	Tudor Engineering Co., for FEMA	H-4025 Amendment No.10	April 1979
King County Unincorporated Areas Revision 1	NHC Inc.	EMW-90-C-3134	September 1991
City of SeaTac Revision 1	NHC Inc.	EMW-90-C-3134	September 1991
City of Bothell Revision 1	NHC Inc.	EMW-90-C-3134	September 1991
City of Normandy Park Revision 1	NHC Inc.	EMW-90-C-3134	September 1991
City of Snoqualmie Revision 2	NHC Inc.	EMW-90-L-3134	May 1995

Community	Contractor	Contract Number	Completion Date
King County Revision 3	Harper Righellis Inc, Portland	N/A	May 20, 1996
King County Unincorporated Areas Revision 4	Harper Righellis NHC Inc.	EMW-93-C-4152	December 2001
Town of Skykomish Revision 4	Harper Righellis		December 2001
City of Issaquah Revision 4	NHC Inc.	EMW-93-C-4152	September 1995
City of Redmond Revision 4	NHC Inc.	EMW-93-C-4152	September 1995
City of Bothell Revision 5	NHC Inc.	EMW-93-C-4152	April 1994
King County Unincorporated Areas Revision 6	Harper Houf Righhellis Inc.		December 2001
King County Unincorporated Areas Revision 6	Montgomery Water Group Inc.		December 2001
City of Issaquah Rivision 6	Montgomery Water Group Inc.	N/A	August 2001
City of Snoqualmie/City of North Bend/King County Revision 7	Harper Righellis Inc.	N/A	October 2001
City of Snoqualmie Revision 7	Harper Righellis Inc.	N/A	April 2005
City of Issquah/King County Revision 7	Montgomery Water Group Inc.	N/A	August 2001

Community	Contractor	Contract Number	Completion Date
City of Issaquah/King County Revision 7	Concept Engineering Inc.	N/A	N/A
King County Revision 8	NHC Inc. and King County Harper Righellis Inc.	N/A	N/A
City of Renton Revision 8	NHC Inc.	N/A	June 2006
City of Duval Revision 8	NHC Inc.	N/A	June 2006
City of Bothell/ City of Kenmore/ City of Redmond/ City of Woodinville/ King County Revision 9	NHC Inc.	*	*
King County Revision 9	NHC Inc.	*	*
City of Burien/ City of Des Moines/ City of Federal Way/ City of Normandy Park/ City of Seattle/ City of Shoreline/ King	NHC Inc.	E00126E08	*

*Data Not Available

Base map information shown on the FIRM for Revision 9 was derived from multiple sources. Base map files were provided in digital format by King County GIS, WA DNR, WSDOT, and Pierce County GIS. This information was compiled at scales of 1:12,000 to 24,000 during the time period of 1994 – 2012.

1.3 Coordination

The coordination for the original King County FIS was completed in multi-agency conferences managed by the FEMA Consultation and Coordination Officer (CCO). The State of Washington Department of Ecology provided input to establish the study priority and the contracting agency. The King County Division of Hydraulics offered valuable assistance to the USACE and the study contractor, in establishing the scope of the original study, coordinating basic data, and defining approximate floodplain boundaries. Topographic maps at contour intervals of five feet, which served as part of the input for the hydraulic analysis and the location of the floodplain boundary lines, were supplied by the King County Department of Public Works. The county also provided information on certain elevation reference marks.

Contacts with the private engineering firms of Bush Roed and Hitchings, Inc., of Seattle, and Horton Dennis and Associates, Inc., of Seattle, were made during the study to discuss field surveys they had conducted.

Permission to enter restricted areas for field surveys was obtained from the City of Seattle and the Chicago, Milwaukee, St. Paul, and Pacific Railroad.

The final CCO meeting was held at the offices of the King County Public Works Department on June 25, 1976. King County officials objected to the "equal conveyance" floodways that were developed in accordance with FEMA guidelines, wanting to apply more stringent floodway criteria. They were especially concerned about the Snoqualmie River, fearing that the loss of valley storage would increase peak discharges if the fringe were filled.

The initial coordination meeting for the original City of Auburn study was held on April 8, 1976. At this meeting, streams to be studied by detailed methods were identified by representatives of the community, the study contractor, and FEMA. During the course of the work, numerous informal contacts were made by the study contractor with the community for the purpose of obtaining data and base maps.

On March 17, 1978, the results of the work were reviewed at an intermediate coordination meeting attended by representatives of the city, the study contractor, and FEMA.

The results of the original study were reviewed at a final CCO meeting held on December 6, 1978. Attending the meeting were representatives of FEMA, the study contractor, and the city. This study incorporates all appropriate comments, and all problems have been resolved.

The initial coordination meeting for the City of Bellevue study was held in April 1975. This meeting was attended by personnel of the U.S. Geological Survey (USGS), FEMA, and officials of the Bellevue Planning and Storm Drainage Utility Departments. Community base maps were selected and streams requiring detailed study were identified.

A search for basic data was made at all levels of government. Topographic maps with a 5-foot contour interval were supplied by the

Bellevue city engineer; these served as preliminary work maps on determining the location of floodplain boundary lines. Some locations and elevations of bench marks were provided by the city and verified by USGS levels.

During the course of the work by the USGS, flood elevations, floodplain boundaries, and floodway delineations were reviewed with community officials. On April 29, 1977, the results of the work by the USGS were reviewed at a final CCO meeting attended by personnel of the USGS, FEMA, and officials of the Bellevue Planning and Storm Drainage Utility Departments.

The initial coordination meeting for the City of Carnation was held in the Carnation Town Hall on July 29, 1977. At the meeting, flooding sources for the City of Carnation were defined and the areas to be studied were identified. Representatives from the City of Carnation, CH2M Hill, Inc. (the study contractor), and FEMA attended the meeting.

Throughout the study, coordination was maintained with the USACE, King County hydraulics division, town officials, Sammamish Valley newspaper, Carnation Planning Commission, and King County Planning Commission. All were contacted to provide information pertinent to this FIS.

The results of the original study were reviewed at a final CCO meeting held on December 19, 1978. Attending the meeting were representatives of FEMA, the study contractor, and the city. No problems were raised at the meeting.

The initial coordination meeting for the City of Des Moines was held on August 19, 1977. This meeting was attended by representatives of the study contractor, FEMA, and the city. This meeting was held to identify areas requiring detailed study and to familiarize city officials with all aspects of the study and to solicit pertinent information.

The Des Moines city government; the Covenant Beach Bible Camp management; and King County Department of Public Works, Division of Hydraulics, were contracted for the coordination of this FIS.

The results of the original study were reviewed at a final community coordination meeting held on March 26, 1979. Attending the meeting were representatives of FEMA, the study contractor, and the city. No problems were raised at the meeting.

In 1981, the City of Des Moines annexed an area along Puget Sound south of the Des Moines Marina. A detailed wave runup analysis of this area was completed in May 1984. An area west of Pacific Highway South (State Highway 99) between Kent-Des Moines Road and South 252nd Street has also been annexed by the City. The analysis to determine the extent of approximate floodplain boundaries in this area was completed in January 1985 and used to update this study.

The initial coordination meeting for the City of Duvall was held in the Duvall City Hall on July 28, 1977. At the meeting, flooding sources for the City of Duvall were defined and the areas to be studied were identified. Representatives from the City of Duvall, the study contractor, and FEMA attended the meeting.

The King County Department of Public Works, Division of Hydraulics; the Sammamish Valley News; and the Duvall Planning Commission were contracted for information pertinent to this FIS.

The results of the original study were reviewed at a final community coordination meeting held on October 2, 1978. Attending the meeting were representatives of FEMA, the study contractor, and the city. No problems were raised at the meeting.

The initial coordination meeting for the City of Issaquah was held on April 8, 1976. The identification of streams selected for detailed analysis was accomplished at this meeting which was attended by representatives of the community, the State of Washington Department of Ecology, FEMA, and a study contractor who was initially chosen to perform the study but did not finally participate.

During the course of the work numerous informal contacts were made by Tudor Engineering Company personnel with the community for the purpose of obtaining information and confirming data. Previous work by the USACE was reviewed and forms the basis of this study.

On January 27, 1977, the results of the work were reviewed at an intermediate coordination meeting attended by representatives of the City of Issaquah, Tudor Engineering Company, and FEMA. A final coordination meeting held on April 2, 1979, resulted in agreement by the same parties, and this report incorporates resolution of all comments received as a result of coordination activities.

The initial coordination meeting for the original City of Kent study was held on April 8, 1976. Streams to be studied by detailed methods were identified at this meeting, which was attended by representatives of the City of Kent and FEMA.

During the course of work, the study contractor maintained contact with the USACE; the King County Division of Hydraulics; and the City of Kent, Department of Public Works.

On May 29, 1979, the results of the study were reviewed at an intermediate coordination meeting attended by representatives of the City of Kent, the study contractor, and FEMA.

The results of the original study were reviewed at a final community coordination meeting held on April 28, 1980. Attending the meeting were representatives of FEMA, the study contractor, and the city. No problems were raised at the meeting.

On April 8, 1976, the initial coordination meeting for the City of Kirkland was held to determine streams to be studied by detailed analysis. This meeting was attended by representatives of the City, FEMA, and the study contractor who was originally chosen to perform the work but did not finally participate.

During the course of the work, numerous informal contacts were made by the study contractor with the community for the purpose of obtaining data and base maps.

On November 30, 1977, the results of the work were reviewed at an intermediate coordination meeting attended by representatives of the City of Kirkland, the study contractor, and FEMA.

The results of the original study were reviewed at a final community coordination meeting held on May 12, 1980. Attending the meeting were representatives of FEMA, the study contractor, and the City. This study incorporates all appropriate comments, and all problems have been resolved.

In August 1977, the initial coordination meeting for the City of Lake Forest Park was held. Streams requiring detailed and approximate study were identified at this meeting attended by representatives of the study contractor, FEMA, and the City of Lake Forest Park.

Initial contact with the Lake Forest Park City Manager, who is also the Public Works Director, was made in February 1978. The City Manager provided background data in the community and descriptions of flood hazard areas in Lake Forest Park. The King County Public Works Department and the USGS were contacted to provide information pertinent to this Flood Insurance Study for Lake Forest Park.

The results of the original study were reviewed at a final community coordination meeting held on December 12, 1978. Attending the meeting were representatives of FEMA and the study contractor, as well as city officials and interested citizens. No problems were raised at the meeting.

The initial coordination meeting for the City of Normandy Park was held on December 5, 1975. It was attended by representatives of the study contractor, FEMA, and officials of Normandy Park. This meeting was held to identify streams requiring detailed study, to familiarize city officials with all aspects of the study, and to solicit pertinent information.

A search for basic data was made at all levels of government. The City of Normandy park, the King County Zoning and Plans Division, the King County Hydraulics Commission and CH2M HILL, Inc. provided maps and other data used in this study.

On August 6, 1976, the results of the work effort by CH2M HILL Inc. were reviewed at the final CCO meeting attended by personnel of the study contractor, FEMA, and officials of the City of Normandy Park. The comments of the officials were incorporated and the study accepted.

The initial coordination meeting for the City of North Bend was held on July 29, 1977. Streams requiring detailed study were identified at this meeting attended by representatives of the study contractor, FEMA, the State of Washington Department of Ecology, King County, and the City of North Bend.

In March 1981, an approximate study was added to the scope of study as a result of consultation among representatives of FEMA, the City of North Bend, and the study contractor.

The King County Engineering and Public Works Departments were contacted to discuss past flooding problems and to gather available topographic mapping and levee plans along with aerial photographs of recent flooding events. The USACE was also contacted to obtain recently developed hydrologic and hydraulic information pertinent to this Flood Insurance Study. The hydrology presented in this study was coordinated with USACE, the State of Washington Department of Ecology, and the King County Department of Public Works.

On September 22, 1981, the results of the study were reviewed at an intermediate coordination meeting attended by representative of the City, the State of Washington Department of Ecology, FEMA, and the study contractor. No problems were raised at the meeting.

The final coordination meeting was held on September 13, 1982, and was attended by representatives of FEMA, the study contractor, and the City. No problems were raised at the meeting.

The initial coordination meeting for the City of Pacific was held on August 1, 1977. Rivers and drainage ditches requiring detailed and approximate study were identified at this meeting attended by representatives of FEMA, the City, and the study contractor.

The USACE, the USGS, the Washington State Department of Highways, Tudor Engineering, city officials, and local citizens provided information used in the report.

The results of the study were reviewed at a final community coordination meeting held on December 3, 1979. Attending the meeting were representatives of FEMA, the study contractor, and the Pacific City Council and members of the public. As a result of this meeting, an area of moderate flood hazard was added to the map.

An initial coordination meeting for the City of Redmond was held to identify streams requiring detailed study. This meeting was attended by representatives of the City of Redmond, FEMA, and the study contractor. Results of the hydrologic analyses were coordinated with the City of Redmond, FEMA, and Tudor Engineering Company.

During the course of the work, numerous informal contacts were made by Tudor Engineering Company, which conducted the study, with community officials for the purpose of obtaining information and confirming data. Previous work by the USACE was reviewed and forms the basis of this study.

The results of the study were reviewed at the final meeting attended by representatives of the study contractor, FEMA, and community officials. The study was acceptable to the community.

The initial coordination meeting for the original City of Renton study was held on April 8, 1976. Streams selected for detailed analysis were identified at this meeting attended by representatives of the community, the original study contractor, and FEMA.

On July 13, 1979, the results of the work were reviewed at an intermediate coordination meeting attended by representatives of the City, the study contractor, and FEMA.

The results of this study were reviewed at a final community coordination meeting held on May 5, 1980. Attending the meeting were representatives of FEMA, the study contractor, and the City. No problems were raised at the meeting.

The initial coordination meeting for the Town of Skykomish was held on July 29, 1977. Streams requiring detailed and approximate study were identified at this meeting attended by representatives of the study contractor, FEMA, and the Town of Skykomish. Town officials provided background data on the community and descriptions of known flood hazard areas in Skykomish.

The King County Public Works Department, the USACE, and the USGS were contacted for additional information to this Flood Insurance Study.

The results of the original study were reviewed at a final community coordination meeting held on April 21, 1980. Attending the meeting were representatives of FEMA, the study contractor, and the town. This study incorporates all appropriate comments, and all problems have been resolved.

The initial coordination meeting for the City of Snoqualmie was held on May 31, 1978. Streams requiring detailed study were identified at this meeting attended by representatives of the study contractor, FEMA, the USACE, and the City of Snoqualmie. A series of meetings was also attended by the city officials, FEMA, and study contractor representatives to discuss possible floodway alternatives. These meetings were held in March 1979, January 1981, and June 1981, and initially resulted in the selection of an equal conveyance floodway for the study. The requirement for expansion of the study to include additional detailed and approximate study mapping for and expected annexation to the city was discussed at the intermediate community coordination meeting held November 4, 1981, and attended by representatives of the study contractor, FEMA, and the City of Snoqualmie.

At the final community coordination meeting held on August 1, 1983, city officials requested that an alternative negotiated floodway be considered that would more fully meet the city’s needs along with those of the adjacent country jurisdiction and ownerships. A negotiated floodway was developed for and approved by the City, King County, and affected county ownerships by written correspondence received during the period from October 1983 to January 1984.

Results of the hydrologic analyses were coordinated with the USACE, the State of Washington Department of Ecology, and the King County Department of Public Works.

The initial coordination meeting for the City of Tukwila was held on April 8, 1976. Streams selected for detailed analysis were identified at this meeting attended by representatives of the community and FEMA.

During the course of the work, numerous informal contacts were made by the study contractor with the community in order to obtain data and base maps. Data were also obtained from the USACE.

On March 26, 1979, the results of the work were reviewed at an intermediate coordination meeting attended by representatives of the City, the study contractor, and FEMA.

The results of the original study were reviewed at a final community coordination meeting held on December 10, 1979. Attending the meeting were representatives of FEMA, the study contractor, and the city. No problems were raised at the meeting.

Initial community coordination meetings for the revised study for King County, Washington, and the Cities of Auburn, Kent, Renton, and Seattle, all within King County, were held on January 16, 1985, and January 24, 1985. At the January 16, 1985, meeting, representatives of FEMA, King County, the Cities of Auburn, Kent, and Renton, the Washington Department of Ecology, and the study contractor, CH2M HILL, Inc., identified streams requiring detailed and approximate study. Representatives of FEMA, the City of Seattle, and CH2M HILL, Inc., identified streams requiring detailed and approximate study at a meeting held on January 24, 1985. The purposes of the meetings were: (1) to inform the county on its status in the NFIP; (2) to identify existing flooding problems and available pertinent data on flooding in the county and cities, and (3) to reach an agreement on the areas to be studied.

During the course of the study, numerous contacts were made and meetings held with local agencies and community officials to discuss and gather available data on flooding history, methods and preliminary results of analyses, and status of proposed near-term drainage system improvements for those flooding sources under study. The USGS was contacted and requested to provide available flow data and data analyses for the streams being studied and surrounding regional drainages. The USACE and the NRCS were also contacted and asked to provide any data or studies they had that were relevant to flooding caused by the streams under study.

Correspondence with the Washington State Department of Transportation (WSDOT) pertained to proposed plans and timing of drainage structure improvements for Rolling Hills Creek and Springbrook Creek under Interstate Highway 405 (City of Renton). Information was also requested for drainage improvements to State Route 522 and Northeast 195th Street, at their crossings of Little Bear Creek.

The initial meeting was held with King County personnel to request available hydrologic and hydraulic information and accounts of flooding history for the flooding sources under study on December 5, 1985. King County Surface Water Management Division staff were contacted and asked to provide basin planning studies and information on any near-term planned drainage system improvements for the flooding sources under study. Design drawings for two bridges being constructed as part of Soos Creek Park on Big Soos Creek were made available through contacts with the King County Division of Parks and Recreation. The Surface Water Division's maintenance personnel were asked to provide information on the operation of the P1 pumping station on Black River, and on the flooding history of the streams being studied.

Storage floodway concepts for local drainages in the Green River Valley, including Mill Creek (Auburn), were discussed at meetings attended by representatives of King County, the Cities of Auburn and Kent, FEMA, and the study contractor, CH2M HILL, Inc.

Preliminary results of analyses for the Green River and levee freeboard issues were presented and discussed at a public meeting on September 11, 1986, attended by representatives from King County, the City of Auburn, the City of Kent, FEMA, and CH2M HILL, Inc.

City of Kent personnel were asked to provide data for a recent drainage basin study prepared for Mill Creek (Kent). Information on proposed drainage improvements for flooding sources under study in the Cities of Auburn, Kent, Renton, and Seattle were requested in the initial stages of study.

Results of the hydrologic analyses were coordinated with community officials, the USACE, the NRCS, and the USGS.

In March 1987, a coordination meeting for representatives of the USACE, Seattle District, and FEMA was held. An analysis of an upper reach of the Green River, immediately above the reach studied in the 1987 King County restudy, was identified. This study was performed under FEMA's Limited Map Maintenance Program.

The final community coordination meetings were held on December 6 and 7, 1988, and were attended by representatives of FEMA, the USACE, and the county. The study was acceptable to the county.

1.3.1 Revision 1

Miller Creek

Various contacts for information regarding the addition of floodplain data for Miller Creek affecting the unincorporated areas of King County, Washington (Reference 94), and then incorporated Cities of Normandy Park (Reference 11) and SeaTac were made by the study contractor in October, November, and December 1990. Coordination with the regional project office and county and city officials, as well as local residents, produced a variety of information pertaining to flood history, available community maps, and other hydrologic data.

1.3.2 Revision 2

Snoqualmie River

The CCO meeting was not held.

1.3.3 Revision 3

Raging River

The initial CCO meeting was held on October 27, 1993, and was attended by representatives of FEMA, King County, and the consultant.

1.3.4 Revision 4

North Fork Issaquah Creek

The initial coordination meeting to incorporate the results of detailed hydrologic and hydraulic analyses of North Fork Issaquah Creek in the City of Issaquah was held on October 20, 1994, and was attended by FEMA and nhc representatives.

Various agencies contacted for information include: the City of Issaquah and King County Public Works Departments; WSDOT; and the USACE, Seattle District. Local residents and engineers for private developers provided information pertaining to flood history and recent and proposed basin development.

Bear Creek & Evans Creek - The initial coordination meeting to incorporate the results of detailed hydrologic and hydraulic analyses of Bear Creek and Evans Creek in the City of Redmond was held on October 20, 1994, and was attended by FEMA and nhc representatives.

Various agencies contacted for information included: the WSDOT; City of Redmond Public Works Department; KCSWM; King County (Surface Water Management division) Engineering Department; and the USACE, Seattle District. The following engineering consultants, who performed previous hydraulic analyses of Bear Creek, were also contacted for information: CH2M HILL; Montgomery Water Group, Inc.; Alpha Engineering Group, Inc.; Land Tech; and Robert Parrott. In addition, local residents and business owners provided helpful information pertaining to previous flooding and development history along Bear Creek.

Middle Fork Snoqualmie River - The CCO meeting was part of the public meeting hosted by King County on January 26, 1995; FEMA, King County, and the King County consultant (Harper Righellis, Inc.) provided presentations.

South Fork Skykomish River - The initial CCO meeting was held on April 6, 1995, and attended by representatives of FEMA, the consultant (Harper Righellis Inc.), and the community. The information for this study supersedes the data presented for the South Fork Skykomish River through the Town of Skykomish.

The Final CCO meeting was held with the Town of Skykomish on January 13, 1997, and attended by representatives of FEMA, King County, Town Council and the Washington Department of Ecology.

Upper Middle Fork Snoqualmie River - The initial CCO meeting was held on January 24, 1995, and attended by representatives of King County, FEMA, and the consultant (Harper Righellis Inc.).

North Fork Snoqualmie River and Tate Creek - The initial CCO meeting was held on November 13, 1995, and attended by representatives of King County, FEMA, and the consultant (Harper Righellis Inc.).

South Fork Snoqualmie River - The initial CCO meeting was held on January 24, 1995, and attended by representatives of King County, FEMA, and the consultant (Harper Righellis Inc.).

1.3.5 Revision 5

The initial CCO meeting was held on September 21, 1993, and attended by representatives of FEMA and nhc. To acquire information for this revision, nhc contacted the Public Works Department of the City of Bothell; the Surface Water Management Division of Snohomish County; Montgomery Water Group, Inc.; Quadrant Company; Alderwood Water District; Bush, Roed and Hitchings; and the USACE.

1.3.6 Revision 6

Tolt River - A public meeting was held September 13, 1995, to present the proposed floodplain and floodway boundaries. Representatives of King County, the City of Carnation, the consultant, FEMA, and the USACE, Seattle District, attended the meeting along with about 70 residents.

In addition to the September 13, 1995, meeting, there were several subsequent Carnation City Council meetings attended by FEMA and King County representatives and the county's consultant (Harper Houf Righellis, Inc.). Also, there was a LOMR produced by FEMA on May 1, 2002, which entailed City Council meetings attended by FEMA and King County.

Upper South Fork Snoqualmie study - A CCO meeting was held as part of the public meeting hosted by King County on February 4, 1997. The meeting was attended by the county's consultant (Harper Houf Righellis, Inc.) and FEMA representatives.

1.3.7 Revision 7

Snoqualmie River - Revision 7 refers to the Lower Middle and Lower South Forks of the Snoqualmie, plus the Overflow channels in between. This mapping effort began in 1999 with an effort by the USACE as contracted by FEMA. The resultant FIRM dated June 30, 1999, was appealed by King County and the cities of North Bend and Snoqualmie. The USACE then produced a PFIRM dated August 8, 2000, which was also appealed. Then King County, serving on behalf of FEMA, utilized the county's consultant (Harper Houf Righellis, Inc.) to revise the technical study. A technical coordination meeting hosted by King County and attended by the staff of the cities and FEMA was held on May 31, 2001. A City and Agency Coordination Meeting was held on September 26, 2001, and was hosted by King County with FEMA, State, and city representatives in attendance. A public meeting held on November 14, 2001, was hosted by King County and the cities received presentations by the county's consultant and FEMA representatives. The submitted study was approved, but BFEs on the subsequent PFIRM dated November 15, 2002, were incorrectly plotted. Therefore, FEMA reissued the PFIRM dated March 28, 2003.

The Final CCO was held on June 16, 2003. This study became effective on April 19, 2005.

Issaquah Creek - The scope of the re-mapping project for the flooding on Issaquah Creek was determined at meetings attended by representatives of the City of Issaquah, King County, and FEMA, on January 12 and March 28, 2000.

The results of the restudy were reviewed at the final CCO meeting held on January 8, 2003. All problems raised at that meeting have been addressed in this restudy.

1.3.8 Revision 8

Patterson Creek - A study kickoff meeting was held October 27, 2005, and was attended by representatives of King County and nhc. The study was also coordinated by King County with the Patterson Creek Flood Control Zone District including a pre-study meeting on November 3, 2005, and a presentation meeting on June 22, 2006.

Lower Snoqualmie River - Briefings by King County and its consultant, Northwest Hydraulic Consultants, Inc. (nhc), to City of Carnation staff and council members were provided on March 16, 2004, June 14, 2004, May 3, 2005, and January 3, 2006, with a public meeting hosted by King County on January 25, 2006, which included presentations by the consultant and representatives from FEMA and the Washington Department of Ecology.

Springbrook Creek - No Information is available from the Springbrook CCO meeting.

Cedar River - A public meeting was held on March 13, 2002 for the unincorporated King County portion of the Cedar River flood study, and attended by representatives from FEMA, King County, and the county's consultant (Harper Houf Righellis, Inc.)

Also, please note that the City of Renton requested a LOMR for the incorporated portion of the Cedar River. This LOMR was reviewed and approved by FEMA on February 16, 2007, but FEMA could not issue a LOMR or physical map at that time. This LOMR is incorporated in the 2010 maps.

Green River - This study was completed by nhc under contract to King County Department of Natural Resources and Parks (KCDNRP). The County is a Cooperating Technical Partner (CTP) with nhc for purposes of conducting FISs. However, this study was funded by King County and also received grant funding from the Washington State Department of Ecology through the Flood Control Assistance Account Program. King County provided project management and technical review of all study products. The County also supplied relevant study data including information on past watershed flooding. The study was started in September 2007 and an initial coordination meeting was held on September 27, 2007, with representatives of King County, nhc, the City of Auburn, the City of Kent, the City of Renton and the City of Tukwila. Additional technical coordination meetings were held with the County and City staff on December 6, 2007, January 10, 2008, February 11, 2008, and February 26, 2008. Ecology staff attended the February 26, 2008, meeting. Coordination meetings were also held with FEMA on November 20, 2007, and November 29, 2007.

Draft floodplain data were delivered to FEMA Region X on March 18, 2008, along with a letter of appeal from King County and letters from each of the valley cities supporting the appeal. On September 23, 2009, FEMA's technical reviewer, Michael Baker Jr., Inc. (Baker), provided King County with review comments and requests for clarification. Baker also provided FEMA Region X with a set of questions regarding how to proceed with finalizing the study. Nhc and King County met with FEMA Region X and submitted a coordinated response to all Baker comments on November 23, 2009. Nhc then worked with Baker to resolve any remaining issues related to the floodplain maps. Revised floodplain boundary delineations were submitted by nhc to FEMA and Baker on March 4, 2010, and the final floodway delineation was submitted on March 10, 2010.

1.3.9 Revision 9

Puget Sound - The study was completed by NHC under contract to KCDNRP, Water and Land Resources Division. The County is a CTP with FEMA for purposes of conducting FISs. King County provided project management and technical review of all study products and supplied relevant study data and coordination with

County citizens. The study was begun in December 2009, and an initial coordination meeting was held February 1, 2010, with representatives of King County, FEMA, STARR, and NHC. Additional technical coordination meetings were held with County staff at key milestones during the study process. Coordination also occurred with FEMA staff through meetings, emails and conference calls. King County, FEMA, and the contractor conducted public outreach by providing a kick-off presentation to representatives from the coastal Cities of Burien, Des Moines, Federal Way, Normandy Park, Seattle, and Shoreline, on January 13, 2011. Draft floodplain data were delivered by NHC to King County and presented at a public meeting on July 21, 2011. King County also conducted individual follow-up meetings with several cities including the City of Federal Way on August 10, 2011, and the City of Burien on August 22, 2011.

Sammamish River – The study was completed by NHC under contract to KCDNRP. King County is a CTP with FEMA for purposes of conducting FISs. The County provided project management and technical review of all study products and also supplied relevant study data. The study was initiated in December 2008, and draft study products were presented to the Cities of Redmond, Woodinville, Bothell, and Kenmore on January 27, 2010. The meeting included presentations by representatives from NHC, FEMA, and the Washington State Department of Ecology.

White River - This study was completed by NHC under contract to KCDNRP. King County is a CTP with FEMA for purposes of conducting FISs. The County provided project management and technical review of all study products and also supplied relevant study data including information on past White River flooding. The study was initiated on March 2007, and draft study products were presented to the public at a meeting in the City of Enumclaw on October 22, 2008. The public meeting included presentation by NHC, FEMA, and the Washington State Department of Ecology representatives.

2.0 AREA STUDIED

2.1 Scope of Study

This FIS covers the geographic area of King County, Washington. Please note that the original scope was referring only to the June 1987 revised study and that only specific portions of King County were re-studied.

The areas studied by detailed methods were selected with priority given to all known flood hazard areas and areas of projected development or proposed construction through 1992.

The following streams were studied by detailed methods in the April 19, 2005, revised countywide FIS:

Raging River -	From Interstate Highway 90 to 0.3 mile upstream of the second Upper Preston Road bridge
Green River -	From approximately 0.3 mile downstream of Pacific Highway to its confluence with Big Soos Creek
Black River/ Springbrook Creek -	From confluence with Green River to SW 16th Street
Mill Creek (Auburn) -	From confluence with the Green River to Highway 18 bridges at RM 6.2
Mill Creek (Kent) -	From Highway 167 to limit of previous detailed study at the Earthworks Park stormwater detention facility outlet
Big Soos Creek -	From confluence with Covington Creek to SE 176th Street
Swamp Creek -	From confluence with the Sammamish River to northern King County boundary
Little Bear Creek -	From confluence with Sammamish River to northern King County boundary
Bear/Evans Creek -	From limit of previous detailed study at confluence with Cottage Creek to Paradise Lake
Issaquah/Holder Creek -	From limit of previous detailed study at SE May Valley Road to Highway 18
West Fork Issaquah Creek -	From confluence with Issaquah Creek to SE 128th Way

May Creek -	From Coal Creek Parkway bridge to SE 109 Place
May Creek Tributary -	From confluence with May Creek to 188th Avenue SE
Cedar River -	From Lake Washington to approximately RM 2.1
North and South Forks of Thornton Creek -	From confluence with Lake Washington to Interstate Highway 5
Longfellow Creek -	From SW Brandon Street to SW Thistle Street
Rolling Hills Creek -	Between first and second crossing of Interstate Highway 405

The Middle Green River was studied by detailed methods in the USACE February 1988 report from its confluence with Big Soos Creek to Flaming Geyser Bridge.

The Tolt River was studied by detailed methods in the SCS June 1982 study from approximately 6,300 feet upstream of the Chicago, Milwaukee, St. Paul & Pacific Railroad to 5.5 miles upstream of the Railroad, a reach of 4.3 miles.

The following streams studied by detailed methods were taken directly from previous Flood Insurance Studies covering King County and incorporated areas (Reference 1 to 18).

Snoqualmie River -	From the Snohomish County line to confluence with the Middle Fork Snoqualmie River, a reach of approximately 45 miles
Middle Fork Snoqualmie River -	From a point approximately 2,323 feet downstream of SE 428th Avenue to a point approximately 2,323 feet upstream of Mount Si Road, a reach of 3.37 miles

North Fork Snoqualmie River -	From confluence with the Snoqualmie River to a point approximately 5,914 feet upstream of 428th Avenue SE, a reach of 1.5 miles
South Fork Snoqualmie River -	From confluence with the Snoqualmie River to a point approximately 8,000 feet downstream of 436th Avenue SE, a reach of 3.8 miles. (Note: A portion of the South Fork Snoqualmie River just upstream of the above-referenced detailed study reach is now depicted as approximate 1-percent-annual-chance flooding. This change was made because updated analysis along that reach superseded the detailed analysis and elevations shown on the effective county map (Reference 1)).
Green River -	From its mouth to confluence with Black River and from Flaming Geyser Bridge to a point approximately 7,286 feet upstream of Whitney Road
Springbrook Creek -	From SW 16th Street to a point approximately 1,690 feet upstream of South 228th Street, a reach of 6.32 miles
Mill Creek (Auburn) -	From State Highway 18 to a point 845 feet upstream of 15th Street SW, a reach of 0.72 miles
Mill Creek (Kent) -	From its mouth to State Highway 167, a reach of 4.24 miles
White River -	From a point approximately 4,330 feet downstream of Burlington Northern Railroad to the Muckleshoot Indian Reservation, a reach of 3.38 miles
White River (Left Bank Overflow) -	From confluence with the White River to the Muckleshoot Indian Reservation, a reach of 0.70 miles

Sammamish River -	From its mouth at Lake Washington to the mouth of Lake Sammamish, a reach of 15.3 miles
North Creek -	From its mouth to a point approximately 10 feet upstream of NE 205th Street at the corporate limits of Bothell, a reach of 1.45 miles
Bear Creek -	From confluence with the Sammamish River to confluence with Cottage Lake Creek, a reach of 5.35 miles
Evans Creek -	From confluence with Bear Creek to a point approximately 2,059 feet upstream of 220th Avenue NE, a reach of 4.66 miles
Issaquah Creek -	From its mouth at Lake Sammamish to Southeast May Valley Road, a reach of 8.0 miles
North Fork Issaquah Creek -	From confluence with Issaquah Creek to a point approximately 740 feet upstream of Issaquah Avenue North, a reach of 0.95 miles
East Fork Issaquah Creek -	From confluence with Issaquah Creek to a point approximately 1,711 feet upstream of 3rd Avenue NE, a reach of 0.87 miles
Tibbetts Creek -	From its mouth to a point approximately 4,610 feet upstream of State Highway 900, a reach of 2.3 miles
May Creek -	From Barbee Mill Road to a point approximately 2,535 feet upstream of NE 31st Street, a reach of 2.02 miles
Vasa Creek -	From the corporate limits of the City of Bellevue approximately 2,500 feet upstream from its mouth to a point approximately 225 feet upstream

Cedar River -	From a point approximately 2,629 feet upstream of Interstate Highway 405 to a point approximately 7,920 feet upstream of the Chicago, Milwaukee, St. Paul and Pacific Railroad, a reach of approximately 19 miles
Mercer Creek -	From its mouth to the confluence of Kelsey Creek and Richards Creek, a reach of 12.9 miles
Mercer Creek Right Channel -	Its entire length, a reach of approximately 1.0 mile
Richards Creek -	From confluence with Mercer Creek to a point approximately 380 feet upstream of SE Allen Road, a reach of 2.65 miles
Richards Creek West Tributary -	From confluence with Richards Creek to a point approximately 310 feet upstream of SE 32nd Street, a reach of 3.22 miles
Richards Creek East Tributary -	From confluence with Richards Creek to a point approximately 680 feet upstream of SE 26th Street, a reach of 0.24 miles
Kelsey Creek -	From its mouth to a point approximately 760 feet upstream of SE 16th Street, a reach of 5.08 miles
West Tributary Kelsey Creek -	From confluence with Kelsey Creek to Redmond Bellevue Road, a reach of 1.57 miles
East Branch of West Tributary Kelsey Creek -	From confluence with West Tributary Kelsey Creek to a point approximately 842 feet upstream of 137th Avenue NE, a reach of 0.44 miles

North Branch Mercer Creek (North Valley) -	From confluence with Kelsey Creek to a point approximately 4,862 feet upstream of NE 24th Street, a reach of 1.49 miles
McAleer Creek -	From a point approximately 40 feet upstream of Bothell Way NE to a point approximately 3,340 feet upstream of NE 185th Street, a reach of 2.13 miles
Coal Creek -	From its mouth to the City of Bellevue corporate limits at Interstate Highway 405 and from the City of Bellevue corporate limits 8,250 feet upstream of Interstate Highway 405 to a point 9,690 feet upstream of Interstate Highway 405, a total length of 0.95 miles
Forbes Creek -	From the City of Kirkland corporate limits approximately 1,420 feet upstream from its mouth to a point approximately 496 feet upstream of NE 108th Street, a reach of 5.66 miles
Lyon Creek -	From confluence with Lake Washington to 35th Avenue NE and from a point approximately 80 feet downstream of Ballinger Road to a point approximately 760 feet upstream of Ballinger Road, a total distance of 1.42 miles
Yarrow Creek -	From 116th Avenue NE to a point approximately 1,515 feet upstream of NE 34th Street, a reach of 0.36 mile
Meydenbauer Creek -	From its mouth to a point approximately 520 feet upstream of 102nd Avenue SE, a reach of 0.36 miles
North Fork Meydenbauer Creek -	From confluence with Meydenbauer Creek to a point approximately 830 feet upstream

South Fork Skykomish River -	From a point approximately 1,505 feet downstream of 5th Street to a point approximately 2,693 feet upstream of 5th Street, a reach of 0.8 miles
Maloney Creek -	From a point approximately 100 feet downstream of Burlington Northern Railroad to a point approximately 890 feet upstream of NE Old Cascade Highway, a reach of 0.32 miles
Miller Creek -	From its mouth to a point approximately 2,530 feet upstream of 12th Avenue SW, a reach of 0.86 miles
Walker Creek -	From confluence with Miller Creek to a point approximately 600 feet upstream of 12th Avenue SW, a reach of 0.33 mile
Des Moines Creek -	From its mouth at Puget Sound to a point approximately 1,960 feet upstream
Unnamed Drainageway -	The ponding of an unnamed drainageway in the central business district in the City of Kirkland, between Central Way and Kirkland Way

Approximate analyses were used to study those areas having a low development potential or minimal flood hazards. The scope and methods of study were proposed to, and agreed upon by, FEMA and the community.

2.1.1 Revision 1 – Miller Creek

Detailed methods were used to study 4.0 miles of the study reach extending from Puget Sound upstream to the proposed King County Lake Reba detention facility near State Route 518.

Approximate methods were used to study the 0.4-mile-long Tub Lake Tributary located just upstream of the proposed detention facility. This minor channel is dry except during flood events.

2.1.2 Revision 2 – Snoqualmie River

This revision was done to update the BFE placements shown on the Snoqualmie River from approximately 1,530 feet upstream of State Highway 202 to its confluence with the South Fork Snoqualmie River to match those shown on the published profiles for that reach.

2.1.3 Revision 3 – Raging River

The revised analysis for the study reach of the Raging River from its confluence with the Snoqualmie River to approximately 0.6 miles upstream of Interstate 90 (I-90) (Downstream Reach) was performed by Harper Righellis, Inc.

The revised analyses for the reach from approximately 0.6 mile upstream of I-90 to approximately 0.3 mile upstream of the second Upper Preston Road bridge (upstream reach) were performed by FEMA.

Prior to this revision, the reach of the Raging River from its confluence with the Snoqualmie River to I-90 had not been studied in detail and appeared as an approximate Zone A on the maps. The reach from I-90 to approximately 0.3 miles upstream of the second Upper Preston Road bridge was studied by detailed methods prior to this revision and appeared as Zone AE on the FIRM.

2.1.4 Revision 4

North Fork Issaquah Creek - The study reach extends approximately 1.2 miles, beginning at the confluence with Issaquah Creek and ending at 230th Avenue SE. The study reach of North Fork Issaquah Creek is primarily located in the unincorporated areas of King County, but includes a very short segment that passes through the City of Issaquah at the I-90 interchange.

Bear Creek & Evans Creek - The restudy covers riverine flooding on approximately 4.6 miles of Bear Creek, a tributary to the Sammamish River. The restudy reach extends from approximately 5,000 feet upstream of the mouth at the Sammamish River, at State Route 202, to approximately 250 feet upstream of the confluence of Bear and Cottage Lake Creeks at Avondale Road NE.

The restudy of Evans Creek included detailed hydraulic modeling from its mouth at Bear Creek upstream to River Mile 0.74.

South Fork Skykomish - This study revises the detailed analyses of the South Fork Skykomish River through the Town of Skykomish and incorporates new detailed analyses affecting King County for reaches extending downstream and upstream of Skykomish. The study area begins at the county line for Snohomish and King Counties and extends 13 miles upstream nearly to the confluence of the Tye and Foss Rivers.

Middle Fork Snoqualmie River - This study includes detailed analyses of a 3.9-river-mile reach of the Middle Fork Snoqualmie River and revises detailed analyses and includes new detailed analysis affecting King County. The study area begins 0.35 miles downstream of the Mount Si Road bridge.

North Fork Snoqualmie River - This study includes detailed analyses for the North Fork Snoqualmie River upstream from its mouth for a distance of 2.41 miles affecting King County, revising previous effective detailed analyses, and adding new detailed analyses in the upstream reaches of the study area.

Tate Creek - The Tate Creek study covers 1.6 miles of creek. The approximate analyses based on a range of calculated peak flows were used to determine typical flow depths and widths for various cross sections.

2.1.5 Revision 5

North Creek - The reach of North Creek that was studied for this revision extends approximately 1,000 feet upstream from the North Creek Parkway to the King-Snohomish County line at 205th Street.

Two small streams were identified for study by approximate methods:

> Horse Creek was studied from confluence with the Sammamish River to the Bothell corporate limits.
>
> An unnamed creek that flows north along 96th Avenue Northeast from the Sammamish River for approximately 0.5 mile upstream.

North Creek LOMR - This study has also been revised to incorporate LOMRs issued on March 3, 1995 (Case Nos. 94-10-

053P and 94-10-067P), and July 5, 1995 (Case No. 95-10-41P). The March 3, 1995, LOMR revised FIRM Panel 0007 C, dated March 2, 1994, to show the effects of a private flood protection system along North Creek from just upstream of I-405 to just downstream of Monte Ville Parkway.

2.1.6 Revision 6

Tolt River - This restudy revises the detailed analysis of Tolt River from the confluence with Snoqualmie River through the Town of Carnation and the unincorporated areas of King County to approximately 6.5 miles upstream of the confluence.

Upper South Fork Snoqualmie River - This study was completed by King County and its consultant (Harper Houf Righellis, Inc.). The county study extends over a reach including approximately 4.9 miles of the Upper South Fork extending from the I-90 bridge crossing near the City of North Bend to above the bridge crossing at 468th Avenue.

2.1.7 Revision 7

Snoqualmie River - This restudy covers the Snoqualmie River main stem, South Fork, and Middle Fork of the Snoqualmie River, including overflows from Middle Fork, Ribary Creek, and Gardiner Creek. The Snoqualmie River detailed study covers a reach of approximately 10 miles. The main stem Snoqualmie River study starts at the Meadowbrook bridge and extends upstream 1.5 miles to the confluence of Middle Fork and South Fork. The Middle Fork study reach extends 3.4 miles, starting from the confluence with South Fork, upstream to the Mt. Si Road bridge. The South Fork study reach extends 5.0 miles starting from the confluence with Middle Fork, upstream to the I-90 bridges (Reference 129).

Issaquah Creek - The Issaquah Creek detailed study reaches cover approximately 6.3 miles. Issaquah Creek was studied from the northern corporate limit of the City of Issaquah in Lake Sammamish State Park, to the southern corporate limit, for a reach of approximately 4.7 miles. East Fork Issaquah Creek (East Fork) was studied from the confluence with Issaquah Creek upstream approximately 1.0 mile to I-90. The Gilman Boulevard Overflow of Issaquah Creek was studied from the point of overflow from Issaquah Creek to its confluence with Tributary 0170 approximately 0.6 miles downstream.

2.1.8 Revision 8

Cedar River –Within the Area of City of Renton - This detailed study includes flooding along the Cedar River within the City of Renton. The study reach begins at the river outlet at Lake Washington and extends 5.36 miles upstream to the City of Renton limits at 149th Avenue SE.

Cedar River – Unincorporated Area of King County - This detail study covers 17 miles of Cedar River, beginning at 149th Avenue SE and extends to the Landsburg Road bridge crossing in the unincorporated area of King County.

Kelsey Creek - The Kelsey Creek study reach LOMR begins at the crossing with I-405 at river mile (RM) 0.0 and continues approximately 4.4 miles upstream near the intersection of 148th Avenue NE and NE 6th Street. The West Tributary study reach begins at the confluence with Kelsey Creek east of the Lake Hills Connector and continues upstream for 0.80 miles. Kelsey Creek and the West Tributary are located within the City of Bellevue, and are part of the Lake Washington watershed. The headwaters of Kelsey Creek originate in the highlands area of Bellevue near Phantom Lake. From there the stream follows a north-northwesterly course approximately 1.8 miles through several pond and marsh areas before reaching the upper extent of the current study reach. The study reach begins flowing north along 148th Avenue NE, but quickly turns northwest and eventually west and south as the stream flows through alternating residential, commercial, and vegetated corridors. The stream continues southward, flowing through the Glendale Golf Course and Kelsey Creek Park before turning west and joining the West Tributary in a broad wetland area located to the east of the Lake Hills Connector. Between the north and southbound lanes of the Lake Hills Connector is the confluence of Kelsey and Richards Creeks. Downstream of the confluence, Kelsey Creek flows through a wetland area followed by an entrenched vegetated corridor until reaching the I-405 culvert. Downstream of I-405, Kelsey Creek flows into Mercer Slough, and finally Lake Washington.

Patterson Creek - This floodplain mapping study comprises an investigation of riverine flooding on Patterson Creek in King County, Washington. The detailed study reach includes approximately 8.3 miles of Patterson Creek starting approximately 0.9 miles upstream of the confluence with the Snoqualmie River and extending to approximately RM 9.2.

Snoqualmie River - The nhc study completed in April 2006 includes the lower 39 miles of the Snoqualmie River. The downstream mapping limit of the study is the State Route 522 Bridge crossing over the Snohomish River, approximately 1 river mile downstream of the confluence of the Snoqualmie and Skykomish Rivers. The upstream mapping limit on the Snoqualmie River is at the base of Snoqualmie Falls just downstream of the City of Snoqualmie, approximately 39 river miles upstream of the confluence with the Skykomish River.

The study limit within King County includes 33.5 miles of river reach beginning at the King County boundary to just downstream of Snoqualmie Falls.

Springbrook Creek – This detailed floodplain mapping study along the Springbrook Creek starts from the Black River pumping station to SW 23rd Street (also known as 180th Street) at the Renton and Kent city boundary. The study covers 16,935 feet of Springbrook Creek and 2,492 feet of the SW 23rd Street drainage canal.

Green River - There are two separate studies performed for the Green River. These studies are called Lower Green River and Middle Green River for the purposes of these studies. The Lower Green River detailed study covers approximately 29.5 miles from boundary between the City of Seattle and City of Tukwila and Unincorporated King County near RM 3.85 and extending to State Highway 18 Bridge. Lower Green River study extends from RM 3.85 to RM 33.3. The Middle Green River detailed study reach includes approximately 12.1 miles of the Green River, starting at the downstream side of the State Highway 18 Bridge and extending to near the upstream end of Flaming Geyser State Park.

2.1.9 Revision 9

Puget Sound - This floodplain mapping study comprised the entire incorporated coastline of Puget Sound in King County, Washington. Backwater effects were adjusted on Des Moines Creek, Miller Creek, and Walker Creek.

Sammamish River – This floodplain mapping study comprises an investigation of riverine flooding on the Sammamish River in King County, Washington. The study area covers the entire Sammamish River, beginning at the source at Lake Sammamish and extending approximately 14 miles upstream to Lake Washington. The work was performed using detailed hydrologic and hydraulic analysis methods approved by FEMA (Reference 190). Also backwater effects from Sammamish River on Bear Creek, North Creek Little Bear Creek, and Swamp Creek were updated.

White River – This floodplain mapping study comprised an investigation of riverine flooding on the White River in King County, Washington. The area of study covers approximately 6.6 miles of the White River, beginning downstream of the State Highway 410 Bridge at RM 22.0 and extending to the outlet works of Mud Mountain Dam at RM 28.6. The work was performed using detailed hydrologic and hydraulic methods approved by FEMA.

The following tabulation presents Letters of Map Change (LOMCs) incorporated into this countywide study:

LOMC	Case Number	Date Issued	Project Identifier
LOMR*	03-10-0047P	1/22/2004	University of Washington
LOMR*	08-10-0762P	03/08/2010	Stage 2- Bear Creek Overflow
LOMR*	11-10-0014P	03/24/2011	North Creek CLOMR, King/ Snohomish County Washington
LOMR*	11-10-1517P	08/17/2012	South Route 202 Widening Project- Evans Creek

*Letter of Map Revision (LOMR)

2.2 Community Description

King County, located in western Washington, is the largest center of population and economic growth in the State of Washington. Its eastern boundary is along the divide of the rugged Cascade Range, and is bordered on the west by Puget Sound. Contiguous counties related economically, as well as geographically to King County are Kitsap County to the west, Chelan and Kititas Counties to the east, Snohomish County to the north, and Pierce County to the south.

The City of Seattle is the county seat and the largest city in Washington. It is located between Puget Sound and Lake Washington. Seattle is important as a port for foreign trade with Asian and South American countries as well as for domestic shipping with Alaska. The 2010 estimated population of Seattle was 608,660 (Reference 183). The area within the Seattle corporate limits is currently 83.9 square miles.

The City of Auburn is located south of Kent. It is approximately five miles from the shores of Puget Sound and 24 miles south of Seattle. Auburn is bordered by Pierce County to the south and by the Cities of Algona and Pacific to the southwest. Auburn has a community area of approximately 30 square miles, and had a population of 70,180 in 2010 (Reference 183).

The City of Bellevue is located in northwest-central King County, eight miles east of Seattle. Bellevue, Washington's fourth largest city, had a population of 122,363 in 2010 (Reference 183).

The City of Black Diamond is located in south-central King County. The city had a population of 4,151 in 2006 (Reference 183).

The City of Bothell was incorporated in 1909 and is located approximately 12 miles northeast of Seattle. The City of Bothell lies within two counties, King and Snohomish. The city is bordered by the City of Kenmore, City of Woodinville, City of Lake Forest Park, City of Mill Creek, and the City of Kirkland. The estimated 2010 population in the City of Bothell was 33,505 (Reference 183).

The City of Burien was incorporated in 1993 and is located 10 miles south of Seattle. The City of Burien covers 7.4 square miles and is bordered on the west by several miles of scenic Puget Sound shoreline, stretching north to downtown Seattle. The small residential communities of Normandy Park and Des Moines are its neighbors to the south. The estimated 2010 population of the City of Burien was 33,313 (Reference 183).

The City of Carnation, incorporated in 1912, is located in north-central King County, on the east bank of the Snoqualmie River. It is approximately 20 miles east of Seattle. The estimated 2010 population in the City of Carnation was 1,786 (Reference 183).

The City of Covington was incorporated in 1997 and is located in the southeastern portion of King County close to the Puget Sound and with views of Mount Rainier. Covington is easily accessible from Highway 18 and State Route 516. The estimated 2010 population in the City of Covington was 17,575 (Reference 183) in an area of 6.5 square miles. The City is bordered by the City of Kent on the western side, the City of Maple Valley to the east and King County to the north and south.

The City of Des Moines, incorporated in 1959, is located in west-central King County. It is just south of the City of Normandy Park and southwest of the Seattle-Tacoma Airport. It is situated in one of the few areas in southern King County along Puget Sound where the land slopes gently down toward the water. The estimated 2010 population in the City of Des Moines was 29,673 (Reference 183).

The City of Duvall, incorporated in 1913, is located on State Highway 203, on the east bank of the Snoqualmie River, in northwestern King County. It is approximately three miles from the Snohomish County line and seven miles north of Carnation. The city had a population of 6,695 in 2010 (Reference 183).

The City of Enumclaw is located in south-central King County, near the Pierce County line. Enumclaw had a population of 10,669 in 2010 (Reference 183).

The City of Federal Way is located 25 miles south of downtown Seattle and just eight miles north of downtown Tacoma. Federal Way has eight miles of Puget Sound waterfront and is in the southwestern corner of King County. The estimated 2010 population in the City of Federal Way was 89,306. (Reference 183) The City was incorporated in February of 1990.

The City of Issaquah is located in west-central King County, approximately 14 miles east of downtown Seattle. The City had a population of 30,434 in 2010 (Reference 183).

The City of Kenmore is located on the north side of Lake Washington, in the northern part of King County. The estimated 2010 population in the City of Kenmore was 20,460 (Reference 183) in an area of 6.1 square miles. Kenmore is bordered by the City of Lake Forest Park, City of Bothell, and City of Brier. On August, 31, 1998, Kenmore was incorporated, making it the newest city in King County.

The City of Kent is located south of Renton and is within two to five miles of the shores of Puget Sound. The City of Tukwila is northwest of Kent and the City of Des Moines is to the west. Kent had a population of 92,411 in 2010 (Reference 183) and occupies an area of approximately 17 square miles. Most of Kent lies on the 2-mile-wide low-lying valley east of the Green River. The bluff area along the east boundary of Kent is drained by several creeks, including Mill, Springbrook, and Garrison Creeks.

The City of Kirkland is located on the east shore of Lake Washington, off Interstate 405 in northern King county. Kirkland is 10 miles east of downtown Seattle, west of Redmond, and just north of Bellevue. The City was founded in 1888 and incorporated in 1905. The estimated 2010 population was 48,787 (Reference 183) - in an area of 11 square miles. The City is bordered by the City of Redmond on the East, City of Bellevue and Yarrow Point on the South, and King County on the Western and Northern borders.

The City of Lake Forest Park is located in the Puget Sound region of northwest Washington in northwestern King County. The community is part of the suburban area that surrounds the Seattle metropolitan center. Lake Forest Park had a population of 12,598 in 2010 (Reference 183).

The City of Maple Valley incorporated August 31, 1997. The City is 5.8 square miles, located east of Kent and Covington, and north of Black Diamond. The estimated 2010 population was 22,684 (Reference 183).

The City of Medina is located in the Eastside region of King County, Washington. Opposite Seattle, and surrounded on the north, west, and south by Lake Washington, Medina is bordered by Clyde Hill and Hunts Point, as well as the satellite city of Bellevue. The estimated 2010 population was 2,969 (Reference 183). According to the United States Census Bureau, the city has a total area of 4.8 square miles, with 1.4 square miles of land and 3.3 square miles of water.

The City of Mercer Island incorporated on July 5, 1960. It included all the land area of the island with the exception of the 70 acre (280,000 m^2) business district. Just over a month later, on August 9, the business district incorporated as the Town of Mercer Island, wholly surrounded by the City. The two municipalities finally merged on May 19, 1970. According to the United States Census Bureau, the City has a total area of 13.1 square miles, with 6.4 square miles of land and 6.7 square miles of water. Mercer Island is connected to Seattle in the west by Interstate 90, carried by the Homer M. Hadley Memorial Bridge (the fifth longest floating bridge in the world) and the Lacey V. Murrow Memorial Bridge (the second longest in the world). I-90 also connects Mercer Island to Bellevue in the east, over the East Channel Bridge. The estimated 2010 population in the City of Mercer Island was 22,699 (Reference 183).

The Muckleshoot Indian Reservation was established in 1857. The reservation is located within the area of Auburn, Washington, located between the White and Green rivers. The Muckleshoot Indian Reservation has a total area of 6.1 square miles (Reference 183).

The City of Newcastle is located 12 miles east of city of Seattle, bordering to the north is Bellevue, and to the south is Renton. The City was incorporated on September 30, 1994. According to the United States Census Bureau, the City has a total area of 4.5 square miles, and 0.22 percent is water. The estimated 2010 population was 10,380 (Reference 183).

The City of Normandy Park is located on Puget Sound in southwestern King County. It is located west of the Seattle-Tacoma Airport and due south to Burien Lake. Normandy Park had a population of 6,335 in 2010 (Reference 183).

The City of North Bend is located in central King County. It lies in the foothills of the Cascade Mountains, approximately 25 miles east of Seattle along Interstate Highway 90. The City of North Bend had a population of 4,621 in 2010 (Reference 183).

The City of Pacific is located in southwestern King County. It shares common boundaries with the City of Algona to the north and Pierce

County to the south. The City of Pacific had a population of 5,859 in 2010 (Reference 183).

The City of Redmond lies in northwest-central King County. It is approximately 10 miles northeast of downtown Seattle. Redmond had a population of 54,144 in 2010 (Reference 183).

The City of Renton is located in western King County. It is located approximately 11 miles southeast of Seattle just north of Kent and just east of Tukwila. Renton had a population of 58,534 in 2006 (Reference 183).

The City of Sammamish is an Eastside suburb, 19 miles east of Seattle, in King County. It was incorporated in 1999. The estimated 2010 population was 45,780. (Reference 183) Neighboring cities include Redmond to the north and Issaquah to the south. According to the United States Census Bureau, the city has a total area of 18.3 square miles.

The City of SeaTac is an outlying suburb of Seattle, located in the southern section of King County. The estimated 2010 population was 26,909. (Reference 183) SeaTac was officially incorporated on February 28, 1990. According to the United States Census Bureau, the city has a total area of 10.1 square miles; 10 square miles of land and 0.1 square miles of water.

The City of Shoreline is located in Western Washington, 15 miles north of downtown Seattle. Shoreline was incorporated 1995, and is surrounded by the older cities of Seattle, Edmonds, Woodway and Lake Forest Park. Covering 11.74 square miles, Shoreline is Washington's 15th largest city. The estimated 2010 population was 53,007 (Reference 183).

The Town of Skykomish is located in northwestern King County. It is in a narrow valley along the south side of the South Fork Skykomish River and is surrounded by the Snoqualmie National Forest. Skykomish had a population of 198 in 2010 (Reference 183).

The City of Snoqualmie is located in central King County. The City lies near the foothills of the Cascade Mountains, approximately 25 miles east of Seattle along I-90. Snoqualmie had a population of 10,670 in 2010 (Reference 183).

The City of Tukwila is located in west-central King County. It is northwest of Kent and west of Renton. It is approximately 12 miles south of Seattle and 22 miles northwest of Tacoma. Tukwila had a population of 19,107 in 2010 (Reference 183).

The City of Woodinville is located in northern King County east of the Bothell. As of the 2010 census, the city had a total population of 10,938. Woodinville was officially incorporated on March 31, 1993. According to the United States Census Bureau, the city has a total area of 5.7 square miles; 5.6 square miles of land and 0.04 square miles of water.

The population of King County was 1,931,722 as of 2010, with 341,000 residing in the unincorporated areas, mostly surrounding the large population center of Seattle. In most suburban communities and unincorporated areas of west-central King County, a decline in farming and significant transition to residential and industrial/commercial development has occurred. Urbanization has spread up the Green and Cedar River valleys where urban build up now covers more than one-fourth of the basin's land areas. The Sammamish River valley is another site of increased residential and industrial/commercial uses. The Snoqualmie River valley is presently the county's primary district for farming and the dairy industry, but urbanization pressures exist for conversion of those agricultural lands to higher value, more intensive land use.

The climate of King County is predominately a mid-latitude, west coast, marine type. Most of the air masses that reach the Puget Sound area originate over the Pacific Ocean. In late fall and winter these masses are moist and about the same temperature as the ocean surface. Orographic effects caused by lifting and cooling of air masses moving inland results in a wide range of precipitation patterns over King County. Fifty percent of the annual precipitation typically occurs in the four month period of October through January, and 75 percent occurs in the six months from October through March. Below 1,500 feet in elevation, the winter precipitation normally falls as rain, occasionally interrupted by periods of snow. During the warmest summer months, the average afternoon temperatures over the county's Puget Sound lowlands are in the lower 70s, decreasing into the 60s in the mountains. Temperatures reach 85°F to 90°F about 5 to 15 days per year, and extremes up to 100°F have occurred in the lower valleys. In winter, afternoon temperatures over the lowland typically range from 35°F to 45°F. The Japanese Current generally moderates the temperatures of winter, but almost every winter there are a few nights when the temperatures range from 10°F to 20°F, with extremes to 0°F.

All of the watersheds in King County are free from glaciers, unlike many streams in other counties lying between the Cascades and Puget Sound.

The undisturbed land cover in King County is dominated by dense conifer forests, with some grass covered prairie-like areas in the lowlands. However, those lowland areas are interspersed with scattered stands of Douglas fir and Oregon white oak. Scotchbroom and other shrubs and seasonal groundcover are typical of those areas. Fresh water marshes commonly have cover consisting of cattails, rushes, and sedges. Big leaf maple trees and red alder are very common between the foothills and Puget Sound.

The Sammamish River is located in northwest King County between Lake Sammamish and Lake Washington. The channel begins at the outlet from Lake Sammamish at the north end of the lake in Marymoor Park. The river then flows northward through unincorporated King County and the Cities of Redmond and Woodinville. At the City of Woodinville, the channel turns to the west, flowing through the Cities of Bothell and Kenmore, before it terminates at Lake Washington. A significant portion of the river valley between the Cities of Redmond and Woodinville are in King County's Farmland Preservation Program and have stringent deed restrictions protecting agricultural uses and prohibiting other development. Much of the rest of the watershed within the study area, as well as the catchment areas upstream, has been developed for residential, commercial, and industrial uses. Consequently, runoff from precipitation events is higher than in times prior to development.

Inflow to the Sammamish River is largely uncontrolled with the exception of discharges from Lake Sammamish which are controlled by an in-channel weir. This weir is located near the upstream end of the Sammamish River at river mile (RM) 13.3 and was built by the USACE in 1966 as part of a channel improvement project. The purpose of that project was to provide protection against spring floods with a 10-percent-annual-chance of occurrence without causing Lake Sammamish to rise higher than an elevation of 32.6 feet NAVD88 (29.0 feet NGVD29). The project included deepening the river, by approximately 5 feet throughout most of its length, and widening the channel, with the excavated material being placed on the river banks (Reference 1). The weir was modified in

1998 to repair degradation of the structure due to wear and to improve fish passage.

Several tributaries contribute flow to the Sammamish River within the study area. The largest tributaries by area downstream of Lake Sammamish are (Big) Bear Creek at RM 12.3, Little Bear Creek at RM 5.4, North Creek at RM 4.4, and Swamp Creek at RM 0.75. There are numerous other small named and unnamed creeks, drainage ditches, and storm drain outfalls that discharge to the Sammamish River between Lake Sammamish and Lake Washington.

White River is located near the Cities of Enumclaw and Buckley, where the river flows in a relatively straight west-northwesterly path from Mud Mountain Dam until it crosses under the SR 410 Bridge. Much of the watershed upstream of the study area has seen only limited development and land cover in the watershed remains dominated by forest. There are, however, some small pockets of residential development within the study reach.

Flows in the study reach are controlled to a large extent by upstream flood management operations at the USACE Mud Mountain Dam located at RM 28.9. Two significant tributaries contribute flow to the White River. These are Red Creek, which enters the White River at RM 26.8, and Boise Creek which enters at RM 22.6. Numerous other named and unnamed tributaries and ditches contribute flow to the White River. A diversion structure at RM 23.6 allows diversions of water from the White River into a flume and canal system which carry the flow to Lake Tapps. However, the diversion does not have a flood control objective and it is generally not operated during high flow events. For purposes of the current flood study, we assumed that no water was being diverted from the White River to Lake Tapps.

2.3 Principal Flood Problems

Climatic and topographic conditions of the upper Snoqualmie Valley create two distinct high-flow periods each year. In the spring or early summer, the seasonal rise in temperature melts snow in the headwaters and causes increased flow. The other high-flow period, the winter flood, is the most damaging. Winter storms bring in moisture-laden air from the Pacific Ocean and mild temperatures causing snowmelt combine to cause floods of high magnitude and short duration. Most of the major floods have occurred during November, December, January, and February.

Without the protection by flood-control reservoirs, the communities along the free flowing Snoqualmie River and its forks are vulnerable to severe flooding such as occurred in November 1959 and December 1975. The largest known flood in the Snoqualmie-North Bend area occurred on November 23, 1959. As the rivers in the basin swelled on that November day, there occurred a classic example of how wildly a river can change its course. About nine miles east of the City of North Bend, the South Fork cut a new channel on the opposite side of its valley through what was a section of the main cross State arterial, the Snoqualmie Pass Highway. Atop its newly cut southerly bank, described as a steep clay cliff, the former river bed remained. The torrent on the South Fork left countless homes damaged in North Bend and contiguous areas.

The violent turbulence of the Middle Fork washed out principal bridges and left other spans badly damaged. This misfortune left over 50 families stranded for more than a week. Some residents on necessary business, some school children, and carriers of mail and milk treaded lightly by foot across the listing bridges that continued to slip on their supports after the flood.

In the City of Snoqualmie, muddy water swept through many homes leaving a trail of destruction. A portion of a city street sank, developing a large cavity as water collected without a natural outlet. Truckloads of concrete slabs and 58 loads of gravel were dumped into the cavity during the flood to save the road, and to prevent adjacent buildings from being swept away. For the entire night of the flood there was no electrical power in the City of Snoqualmie. This flood had a discharge at the USGS gage near the City of Snoqualmie of 61,000 cubic feet per second (cfs). This discharge is equivalent to a 25-year flood at this point (Reference 21).

The largest known flood in the Carnation area occurred in December 1975. Agriculture and transportation damages constituted the principal losses. However, the lower valley is inundated to some extent almost every winter. Other major floods occurred in February 1932, December 1967, and January 1969.

Storms which cause flooding in the Tolt River Watershed are usually associated with long, steady rains (i.e., winter maritime occluded frontal systems) which are typified by longer duration, more uniform intensity, and more evenly distributed precipitation than the unstable shower (convective) storms. With this type of rainstorm, the flooding in one basin, such as the Tolt, will be associated with flooding on adjacent basins; thus, the rare occurrence of a 1-percent-annual-chance frequency flood on the Tolt would most likely be associated with high water backwater of the Snoqualmie River.

The elevation of future floods depends upon the level of the Snoqualmie River at the peak discharge of the Tolt River, the amount of landfill or diking, the physical arrangement or layout, and the hydraulic conditions of the channel.

High water marks were provided by landowners and field estimates of survey crews. There are no precipitation gages with long records in the watershed, but the Seattle Water Department has eight storage gages established in 1962-67. The average annual precipitation at these locations ranges from 90 inches (228.6 cm) to 157 inches (398.8 cm).

The largest historical flood since 1953 on the Tolt River near Carnation occurred in 1959 with a peak discharge of 17,400 cfs.

The Raging River is characterized by a relatively steep gradient resulting in high-velocity flood flows and significant bank erosion and channel aggradation problems. These characteristics have lead to increased flood levels, based on local resident accounts, most likely caused by reduction in channel floodflow conveyance capacity with aggradation. In past floods, large boulders, logs, and debris have been swiftly transported down the river and have partially blocked bridges and threatened the levee systems in the Fall City area.

The peak recorded published flow at the USGS gage near Fall City during 40 years of gage operation through 1985 is 3,960 cfs. This occurred on January 24, 1984, and was approximately a 35-year event. Although final estimates of peak flows for an event on November 24, 1986, are not available, provisional estimates between 4,400 and 5,300 cfs have been made by the USGS (Reference 22). Based on the existing frequency curve previous to that event, those flows would correspond with greater than a 2-percent-annual-chance event. Flows in excess of 3,000 cfs were also recorded on February 9, 1951, December 3, 1975, and December 15, 1979 (recurrence intervals ranging from 20 to 30 years).

Flooding damage to crops and property in the lower Green River Valley has been a problem since the earliest settlement of the area. Flooding occurred almost annually but the impact to farmland was minimal. After urbanization, the impact of flooding became more severe. Rapid increase in construction of roads, housing, and parking lots increased the volume and rate at which runoff reached the valley floor. Commercial and industrial landfills have been typically located in the lower valley, resulting in alteration of natural drainage patterns and reduction in overbank storage.

During periods of excessive precipitation, surface and subsurface runoff from the steep valley walls cause groundwater elevations in the valley floor to rise significantly. This creates open ponding in topographically depressed areas. This condition is further aggravated by floodflows and corresponding high water elevations on the Green River, resulting in a perched channel condition, which prevents natural drainage of subsurface water. In some areas, the overlying soils are generally less pervious than the deeper sands and runoff collects in pond perched above the water table.

The land in the lower Green River Valley from Auburn to Renton had historically been inundated by large floods, such as occurred in December 1933, November 1959, and February 1951, until the construction of the Howard A. Hanson Dam. Since operation commenced in 1962, the dam, in combination with levee systems constructed along river segments below Auburn, has prevented that degree of flooding and limited flood damages. During the floods of January 1965, December 1975, and December 1977, discharges downstream were effectively reduced to non-damaging levels. The 1977 flood would have had the highest unregulated peak of any event since diversion of the White River in 1906 (Reference 23).

The USACE is responsible for regulation of dam outflows to a rate that will limit flows at Auburn, together with local inflows below the dam, to 12,000 cfs for up to a standard project flood frequency. This flow rate represents a 2-year recurrence interval flood event on the unregulated discharge frequency curve (Reference 24).

Under regulated conditions, significant flooding still does occur in areas unprotected by levee systems and from interior local drainage runoff to the Green River. High water levels in the Green River and concerns with existing levee system freeboard and structural integrity limit the discharge of runoff waters carried by Mill Creek (Auburn), the Black River, and various other tributaries. The high water levels of the Green River require that the tributary flows be stored and released by gravity or pump discharge to the river channel in a manner consistent with the requirements of the Green River Management Agreement (Reference 25). Under existing conditions, extensive backwater flooding occurs at the uncontrolled outlets of Mill Creek (Auburn) and Mullen Slough, south and west of State Routes 516 and 167, respectively.

The P1 pumping station pumps the flow from the Black River into the Green River. The firm capacity of the pumping station is significantly less than the peak inflows from Springbrook Creek estimated to reach it. No major backwater effects and associated flooding of overbank areas has occurred (Reference 26) since the pump station construction in 1972 and later P1 storage pond excavation. However, analysis shows that backwater flooding will occur upstream of the pump station under existing inflow runoff assumptions and hydraulic structure conditions. Peak outflows from the pump station have not exceeded 525 cfs (November 1986 event) with nominal P1 pond storage (Reference 26).

Flooding from Mill Creek (Kent) drainage, downstream of the Earthworks Park regional stormwater detention basin, results primarily from limited capacity hydraulic structures and low stream gradients, extending downstream to its discharge to Springbrook Creek. Downstream of James Street, east bank overflow will occur at peak flood stages of Mill Creek and flow to the headwaters of Springbrook Creek. Although no stream gage records exist for Mill Creek, outflow from the Earthworks Park detention basin for the January 1986 storm event was estimated to be approximately 90 cfs, computed from surveyed high water mark data and hydraulic rating of the outlet.

Flooding in the Mill Creek (Auburn) drainage is caused by backwater effects from the Green River, and by overburdened channel capacities and restrictive hydraulic capacities at various roadway culvert crossings. During times of high flood stages on the Green River, which can extend from a few days up to a 1-week period for an extreme storm event, storage of Mill Creek floodwater along the valley floor behind the leveed Green River occurs. A portion of the flow, which would normally enter the Green River via Mill Creek, overflows into Mullen Slough for release back to the Green River, as it recedes, at a lower (downstream) hydraulic gradient.

No continuous stream gage records exist within the Mill Creek basin. Crest stage gage records between 1950 and 1970 on the Peasley Canyon tributary drainage indicate a peak recorded discharge of 112 cfs in February 1951 (Reference 27). Mill Creek peak runoff for the January and November 1986 runoff events was not considered extreme based on local accounts and field reconnaissance of extent of flooding.

Flooding along Big Soos Creek is primarily limited to the lower gradient channel reaches to the mid to upper portion of the basin, extending upstream from Kent-Black Diamond Road. Wide marshlands are typical in those reaches with narrow channels with limited hydraulic capacities. Existing restrictive bridges and other channel constructions result in increased flood levels and corresponding flooding of the low-lying overbanks. Development does not currently encroach significantly on the floodplain.

The maximum recorded floodflow for Big Soos Creek for the 25-year period of record at the USGS stream gage station located above the fish hatchery near the Green River is 1,090 cfs. That event occurred on February 28, 1972, and has an approximate recurrence interval based on period of record frequency curve computation of less than 10 years. Floodflows of greater than 1,000 cfs also occurred in November 1960, January 1964, and February 1982. Preliminary estimates of peak flows for the January and November 1986 storm events do not exceed 900 cfs.

On the White River, the flood of 1975 overtopped and subsequently eroded a section of the levee on the left (south) overbank, upstream of the study area at approximately RM 10.6. It is unlikely that the levee will be repaired within the foreseeable future. Consequently, high flows on the White River are expected to cause flooding in the left overbank, outside the levee, for a distance of approximately 2.6 miles before floodwaters are returned to the main channel at approximately RM 8.0. Approximately 0.8 mile of this overbank flooding occurs within the Auburn corporate limits, inundating areas which are presently wooded and unclassified, but which are earmarked for future single-family residential development.

The amount of storage provided naturally by Lake Sammamish has a moderating influence on flow, and the channelization project by the USACE has significantly reduced flood problems on the Sammamish River. The primary areas that are subject to flooding are adjacent to tributary inlets where the channel berm is interrupted.

On Lake Sammamish, the highest flood during a 37-year period of record occurred on February 11, 1951, when the water-surface of the lake reached an elevation of 33.44 feet National Geodetic Vertical Datum (NGVD). Calculations by the USACE indicate that the 1951 inflow would have raised the lake elevation to 29 feet NGVD had the present improved outlet been in operation (Reference 28). On December 5, 1975, the lake level reached 29.70 feet NGVD. Generally, the lake level ranges between 25 feet NGVD in summer and 28 feet NGVD in winter.

The largest recorded floodflows on Swamp Creek occurred on January 18, 1986, when a flow of 1,090 cfs (provisional) was measured at the USGS gaging station at Kenmore. This flow exceeds the 1-percent-annual-chance event magnitude based on the 23 years of gage record through 1986. The previous measured peak flow on Swamp Creek occurred on March 6, 1972, with a value of approximately 490 cfs.

Numerous private bridges along the lower reaches of Swamp Creek and encroachment on the creek channel from development provides restrictions to flow that may result in increased flood levels and additional overflows to typically low-lying overbank areas. Although localized flooding damages were reported for the January 1986 extreme runoff event, they were primarily related to channel bank erosion, overtopping of roadways and resulting damages (including culvert washouts), and limited damages to residential structures.

The natural channel of North Creek lies on the opposite side of the valley from where the stream now flows. The creek was relocated to the high side of the valley to improve its capacity. The last reported flooding on North Creek occurred in March 1950, when the flow reached 680 cfs. This event was slightly greater than then 1-percent-annual-chance recurrence interval. Because land use in the valley is agricultural, the flooding had minimal impact. Highwater in December 1975 was reportedly contained within the North Creek channel. There are no gage records of this event. Localized ponding areas develop every winter because of the poorly drained soils in the valley.

Frequent flooding occurs on Little Bear Creek in the Woodinville area near the confluence of the Sammamish River. The hydraulic structures and channel capacities are limited along the stream reach between the culverts under NE 178th Street and State Route 202. This causes frequent overflows, primarily along the south bank, which are removed from the stream system and flow independently to the Sammamish River. South overbank flows, downstream of the State Route 202 culvert, combine with overflow immediately upstream of the same culvert and flood the low-lying Burlington Northern Railroad underpass area with ponding depths exceeding six feet. This overflow and ponding, with outflow across NE 175th Street south to the Sammamish River, frequently floods local commercial structures. Limited overflows along the north creek bank, upstream of NE 178th Street, cause shallow flooding to commercial structures and surrounding roadways, as was experienced in the January 1986 event. Flooding damages upstream of State Route 202 are not typically severe, primarily because of the undeveloped character of areas near the stream course and floodplain.

No operational stream gages exist on Little Bear Creek to directly estimate flooding magnitudes; however, analyses of hydraulic ratings for the channel, culvert, and overflow components provided an approximate peak flow estimate of 650 cfs for the January 1986 event. Review of local precipitation records and comparison with, and transfer of, flow records from adjacent gaged basins indicates that the event most likely represented a recurrence interval of greater than 1-percent-annual-chance magnitude. A private commercial business crossing between the State Route 202 crossing and NE 178th Street was washed out during that flood event.

The flood season for Bear and Evans Creeks is from October to March. The greatest floods are caused by rainstorms although melting snow may occasionally augment flooding. Storm runoff in the Bear Creek basin is comparatively slow because of the moderate terrain, the unimproved condition of the channels, and the small amount of residential and commercial developments in the watershed. As a rule, the stream rises to a peak stage within a day and the duration of flooding is less than a week.

The largest recorded floodflow on Bear Creek within the limited period of gage record was a recent event on January 18, 1986, with estimated provisional peak flows of 390 cfs at the USGS gage near Redmond (upstream of Cottage Lake Creek) and 1,550 cfs at the USGS gage at Redmond, upstream of the Sammamish River confluence. Based on updated frequency curves including that event, the estimated recurrence interval of floodflows within the Bear Creek basin for that event is approximately 40 to 50 years. The previous recorded peak flows at those gages were 250 cfs and 456 cfs, respectively, although the gage record is limited to eight years for each station.

There are numerous bridges over Bear Creek within the study area, many of them private crossing with restrictions that limit capacities and increase upstream flood levels. During major floods, debris collecting at these structures may significantly increase the extent of flooding and potential for overflow with resulting damages to roadways and adjacent structures. Damage reports from the January 1986 event were not extensive; however, roadways were overtopped at a few crossings and a mobile home park was flooded and had to be evacuated along the lower reaches of Bear Creek.

The flood season for Issaquah and Tibbetts Creeks is during the winter from October to March. The greatest floods are caused by rainstorms although melting snow occasionally augments flooding. The creeks rise quickly during heavy rainfall because of the steep terrain in the watersheds. As a rule, the streams rise to crest stage within a day and the duration of flooding is less than a week.

The largest recorded peak floodflow on Issaquah Creek in the years of USGS gage record since 1964 occurred on November 24, 1986, when a peak discharge of 3,050 cfs (provisional) was recorded at the USGS gage "near mouth, near Issaquah" (Reference 22). That floodflow represents an approximate 25-year recurrence interval based on frequency curves for gage record prior to that event. The flooding event of January 18, 1986, produced the third highest period of record gage flow on Issaquah Creek, estimated at 2,400 cfs (provisional) by the USGS (Reference 29), with an estimated recurrence interval of less than 10 years. Peak runoff for a January 1, 1964, event of 2,870 cfs represents the second highest flow on gage record.

There are numerous bridges spanning Issaquah Creek. The clearance and flow capacity of many of these bridges are restricted. During major floods, debris collecting at these structures may significantly increase the extent of flooding. Development along Issaquah Creek has encroached on the channel, particularly in the downstream reaches in and surrounding the

City of Issaquah. This encroachment reduces the flood-carrying capacity of the channel, increasing the flood depths in adjacent areas. Local accounts and aerial photographs (Reference 29) of flooding in the City of Issaquah and along the West Fork Tributary indicated that flood levels for the November 1986 event were the highest in recent years. Numerous roads and structures were inundated. Peak floodflows from the West Fork of Issaquah Creek are relatively small compared to those of the mainstem; however, significant areas of flooding occur in the upper reaches of that tributary. The flooding is a result of an extremely low gradient stream channel, having a small channel capacity with wide and flat overbanks.

Flood damage on May Creek occurs mainly at the mouth where a lumber mill was built on the small delta there. Upstream of I-405, May Creek flows generally within a canyon. Flooding problems in this reach are the result of surface runoff and ground-water seepage from the steep canyon walls rather than excessive overflow of May Creek.

For the reach of May Creek under study upstream of the Coal Creek Parkway, flooding results from channel and bridge capacities restrictions and flattening of stream gradients in the upper May Valley area. For the reach extending upstream to 146th Avenue SE, flooding is typically confined to a relatively narrow, steep channel. Upstream from that crossing, the floodplain expands to the overbanks where floodplain inundation widths between 500 and 1,000 feet are typical for significant storm events. Filling of floodplain overbanks and reduction in storage, and debris buildup at the hydraulic structures, can increase flood levels and the extent of upstream overbank flooding. Flooding extent on the May Creek Tributary, upstream of SE May Valley Road, results primarily from backwater effects of the main channel at their confluence.

A USGS stream gage exists on May Creek (discontinued) at its mouth near Renton. The peak flow recorded at that station during the 15 years of gage operation was 510 cfs on December 3, 1975. This corresponds to a storm with a recurrence interval of approximately10 to 15 years based on the period of record frequency curve. High water marks located immediately upstream and downstream of the gage were observed for the January 1986 storm event. Results of approximate rating analyses at the gage for that event indicated floodflows potentially exceeding 800 cfs with an expected recurrence interval of greater than the 2-percent-annual-chance flood. Flooding, including inundation of structures in the upper May Valley area, was reported for that event.

Flooding along Vasa Creek generally occurs during the winter months, November through February, when storms originating over the Pacific Ocean bring intense precipitation. These storms usually last two or three days, and streams may increase from low flow to flood discharge within 6 to 12 hours. The major flood problems are those of inundation and damage of private property from out-of-bank floodwaters, primarily along low gradient reaches of the streams.

The Cedar River is subject to frequent flooding damages, particularly in its upper reaches, beginning with minor flooding and bank erosion when the river flow, measured at Landsburg, exceeds 2,500 cfs. This magnitude of flows typically occurs annually. Major flooding occurs when river flows reach 4,000 cfs, which happens on the average once every five to 10. years. Topographic and climatic conditions of the basin produce two high-water periods during the year. The highest flows normally result from extreme rainfall and the accompanying snowmelt that can occur during the late fall and early winter. Flooding can also occur during spring months, resulting primarily from snowmelt events.

Stream flow on the Cedar River has been recorded almost continuously since 1895 at the gage near Landsburg. The greatest flood which has occurred over the past 50 years took place on December 4, 1975, with a peak discharge at Landsburg of 8,800 cfs. Based on an updated frequency curve for the Renton USGS stream gage for the 40 years of record through 1985, the recurrence interval for that event exceeded 1-percent-annual-chance. Preliminary peak flow estimates by the USGS (Reference 22) for the recent November 1986 event indicate a peak flow of approximately 5,300 cfs, with a recurrence interval of approximately 100 years. Preliminary peak flow estimates by the USGS (Reference 22) for the recent November 1986 event indicate a peak flow of approximately 5,300 cfs, with a recurrence interval of approximately 10 years.

Damages in the Cedar River basin from the December 1975 flood event were estimated at $1,760,000. In the reach under study, the west bank of an improved channel at the mouth of the Cedar River was overtopped above the South Boeing Bridge and the Renton Municipal Airport experienced significant flooding and had to close down until the floodwaters receded. Extent of flooding for the November 1986 event in the lower 2-mile reach under study was mainly limited to the improved channel with the exception of some overbank flooding adjacent to the Renton Airfield. Upstream of the improved channel, portions of the Maplewood Additions and other scattered residential developments have been inundated by past flooding events. Log and debris jams have been experienced on the lower river channel, especially during the 1933 and 1975 floods.

The lower reach of the river channel, through the City of Renton, has been aggrading in recent years based on comparison of current and previous cross-section data. This may result in increases in flood levels and potential overflows.

A reach of the Cedar River about 0.8 miles in length along the right bank immediately upstream of I-405 is seriously obstructed. Various private enterprises along this river reach have encroached on the stream bed by dumping waste concrete and asphaltic concrete. Fill has been placed, paved, and riprapped to accommodate parking facilities for tenants residing at the Riveria Motel. This fill encroaches into the river 25 to 40 feet along the entire width of the property. Encroachment of this type reduces the river channel capacity, creating higher water levels adjacent to and upstream of these areas.

Flooding along Mercer Creek, Richards Creek and its tributaries, Kelsey Creek and its tributaries, and North Branch Mercer Creek generally occurs during the winter months, November through February, when storms originating over the Pacific Ocean bring intense precipitation. These storms usually last two to three days and streams may increase for low flow to flood discharge within 6 to 12 hours. The major flood problems are those of inundation and damage of private property from out-of-bank floodwater, primarily along low-gradient reaches of the storms.

Ice jams have little impact on flooding when culverts and bridges are free of debris. Flood elevations, however, are increased because of the limited capacity of some culverts. In some cases, this limited capacity is intended as a means of peak-flow retention.

Numerous bridges and culvert systems exist along Thornton Creek from its outlet to Lake Washington at Matthews Park to its forks, and extending upstream to and above I-405. Flooding for moderate runoff events is primarily contained by the Thornton Creek drainage system. However, the restrictions imposed by the crossings and encroachment on the channel in this heavily urbanized basin result in backwater flooding and overflow of channel banks and structures, with resulting damages, under more severe runoff conditions. Debris collection, particularly as it affects outflow to the diversion works, has had significant impacts on increasing inundation levels during past flooding events. Since the November 1978 storm event that resulted in flooding problems augmented by debris, the City of Seattle has improved the operation and maintenance of the diversion works structure, located at RM 1.3 on Thornton Creek, below the confluence of the North and South Forks. This diversion works structure diverts flows up to an estimated 340 cfs for the 1-percent-annual-chance event into an abandoned 72-inch concrete sewer pipe. This pipe discharges directly into Lake Washington just north of Matthews Beach. The diversion structure functioned adequately during the January 1986 storm event. Based on hydraulic rating analyses performed from surveyed high water marks, peak runoff for that event was estimated at 560 cfs above the diversion and downstream of the creek forks and 220 cfs in the main channel downstream of the diversion works.

Some minor flooding has occurred in the past in the lower reaches of McAleer Creek. This flooding was caused by hydraulic structures of inadequate capacity or sedimentation and debris accumulation. Particular dates of past flooding are not available.

Flooding along Coal Creek generally occurs during the winter months, November through February, when storms originating over the Pacific Ocean bring intense precipitation. These storms usually last two to three days, and streams may increase from low flow to flood discharge within 6 to 12 hours. The major flood problems are those of inundation and damage of private property from out-of-bank floodwaters, primarily along low gradient reaches of the streams.

The flood season for Forbes Creek in the lower Puget Sound region is normally during the winter from October to March. The larger floods are caused by rainfall, although melting snow occasionally augments flooding.

Forbes Creek has no gaging station and there is no written record of historical flooding. Discussions with residents revealed a history of localized flooding of short durations caused by brief periods of intense rainfall.

Debris collecting at structures and residents encroaching on the channel capacity by placing various types of materials to stabilize the streambank, may significantly increase the extent of flood.

Flooding along Lyon Creek has occurred in the lower reaches and also in the southwest corner of NE 185th Street and 35th Avenue NE nearly every winter. Hydraulic capacity has been greatly reduced in the two concrete box culverts under Bothell Way Northeast. Sedimentation in the southern portion, up to approximately 2 feet from the original invert, has diverted all the flow through the northern portion. At higher flows this would create unnecessary backwater in the upstream channel in front of the shopping center complex or the sediment could become dislodged causing a blockage elsewhere downstream.

Flooding along Yarrow Creek, Meydenbauer Creek, and North Fork Meydenbauer Creek generally occurs during the winter months, November through February, when storms originating over the Pacific Ocean bring intense precipitation. These storms usually last two to three days, and streams may increase from low flow to flood discharge within 6 to 12 hours. The major flood problems are those of inundation and damage of private property from out-of-bank floodwaters, primarily along low-gradient reaches of the stream.

The major source of flooding within Skykomish is the South Fork Skykomish River. Flooding occurs primarily during the winter due to rainstorms which bring intense precipitation and are accompanied by warm winds that rapidly melt the accumulated snowpack. During such storms, river discharges may increase from a relatively low base flow to near flood stage within a few hours.

Residents report that the largest flood on record occurred in November 1959. The return period for that flood is approximately 30 years. Although a dike contained most of this flow in the eastern part of the town, water covered the central and western areas. A flood also occurred in 1975, and floodwaters reached the tops of the levees. The return period for that flood is less than three years.

The other potential source of flooding within Skykomish is Maloney Creek, which meets the South Fork Skykomish River near the western corporate limits. This stream flooded in 1933 when a logjam that had been holding back the flow broke. No information on the recurrence interval for this flood is available. There has been no flooding reported on Maloney Creek since that time.

The flooding problems in the lower portions of Miller and Walker Creeks are a result of increasing development, which has caused more rapid runoff in those creeks. This development is primarily outside the City of Normandy Park boundary and has been the subject of much discussion and some litigation. Damage has generally been limited to stream erosion and some limited flooding around residences.

The area most subject to flooding along the lower portions of Des Moines Creek is owned by the Covenant Beach Bible Camp.

The streamflow of Des Moines Creek exceeds the channel capacity several times each year, resulting in several thousands of dollars of damage each year. Damage is usually limited to bank erosion, overbank deposition, and some shallow flooding in and around occasionally occupied camp cottages.

The last major flood event along Miller and Des Moines Creek was in February 1972, and had a recurrence interval estimated at 10 years. As a result of Miller Creek flooding, a suit was brought against the county to restrict the diameter of the 8.5-foot culvert on First Avenue South through which Miller Creek passes. A 6-foot diameter collar was placed in the upper end of the culvert. The effects of the collar have been included in the hydraulic analysis of Miller Creek. As a result of Des Moines Creek flooding, a 4-foot-deep hole was eroded around one of the cottages and water up to approximately 2 feet deep was standing in others. The December 15, 1977, high tide provided a high tide in Puget Sound of an approximate recurrence interval of 70 years. This high tide was accompanied by very little wind.

Flooding occurs at numerous locations along Longfellow Creek because of restricted channel and culvert capacities and partial obstruction of the natural channel because of debris accumulation. Overtopping of the majority of the roadway crossing between SW Brandon Street and SW Myrtle Street, including localized flooding of properties, structures, and bank erosion, occurred during the January 1986 flooding event. Downstream of the study limit, flows at SWE Nevada Street overtopped an approximate 30-foot-high roadway fill, partly because of culvert debris blockage, and resulted in failure of that crossing with extreme floodflows released to the downstream drainage. Surface flooding also occurs at locations where the lateral storm drainage systems have insufficient capacity to convey storm runoff into Longfellow Creek.

The existing culverts that convey Rolling Hills Creek under I-405 at its intersection with State Route 167, and through a closed culvert behind the Renton Cinema, cause overbank flooding north of the channel in the parking areas for the Cinema and the Renton Village Development. Significant reduction in peak flows through the downstream highway culvert is achieved from routing of floodwater that pond in the overbank.

2.3.1 Revision 1 – Miller Creek

On January 8, 1990, a flood on the order of the 1-percent-annual-chance event inundated farm lands, pasture lands, and residential yards neighboring the creek. Farm and pasture lands sustained no significant damage, but several homes did. A homeowner located at the northwest corner of South 160th and 9th Avenue South reported 4 feet of water in her basement. The yard of the home located on the southwest corner of this intersection was severely eroded by high-velocity water issuing from the culvert that conveys Miller Creek flow under 160th Avenue. Near 8th Avenue South, the stream jumped its west bank and damaged the contents of a garage/workshop. Several homes between 8th Avenue South and Des Moines Way were also flooded.

Downstream from 1st Avenue, the creek is confined to a deep ravine, which does not pose a threat to neighboring property. As it leaves the ravine, the creek flows along the west side of the Southwest Suburban Sewer District sewage treatment plant.

During the January 1990 flood, the creek remained within its banks through this reach. Below the treatment plant, the stream profile begins to flatten and the floodplain widens. Two homes at the intersection of Miller Creek and SW 175th Place were flooded. Below SWE 175th Place, the floodplain widens and has been preserved as a community park for residents of the City of Normandy Park. Much of it was also covered by water during the flood.

2.3.2 Revision 2

Snoqualmie River

No additional information is available.

2.3.3 Revision 3

Raging River

The Raging River experienced a flood of 6,220 cfs (nearly the 100-year event flow) in November 1990, and 5,330 cfs in November 1986. Both events significantly impacted Preston-Fall City road and numerous properties and structures.

2.3.4 Revision 4 – North Fork Issaquah Creek

Information on the frequency and extent of past flooding along North Fork Issaquah Creek is very limited, and no information is available for most of the study reach. Areas where past flooding has occurred were identified through interviews with local residents during field surveys made by nhc in October and November 1994, and during a 2-year flood event in February 1995.

At Issaquah Creek near the mouth, a 56.6-square-mile basin area, major floods with nearly identical peak flows, 3,100 and 3,200 cfs, were recorded on November 24, 1986, and January 9, 1990, respectively. These floods each had a return period of approximately 30 years. Major floods are believed to also have occurred on North Fork Issaquah Creek on the same dates. Coincident flooding is confirmed by a King County stream gage on the North Fork Issaquah Creek channel to have occurred on January 9, 1990; that gage had not yet been installed at the time of the 1986 flood.

No information on past flooding was available for the 0.7-mile reach between the 60th Street SE bridge and the I-90 interchange. Flooding of the area immediately upstream of the I-90 interchange occurred on several occasions after construction of this interchange in approximately 1968.

Additional information on past high-water levels is available from a stream gage operated by the King County Division of Surface Water Management (KCSWM) at the 66th Street SE bridge crossing of North Fork Issaquah Creek. The maximum water elevation recorded during the January 9, 1990, flood was 72.8 feet, which is approximately 0.6 foot below the low cord of the bridge.

2.3.5 Revision 5

North Creek

No additional information to add.

2.3.6 Revision 6

Tolt River

Recent major floods on the Tolt River occurred in November 1990, November 1995, and February 1996. Each had a peak magnitude that was smaller than the current 10-year flood. The three largest Tolt floods on record occurred in 1950s before dam operation began, and all three were close to the current 20-year flood.

The Tolt River levees were damaged at three locations in the winter of 1995/1996, and were subsequently repaired. The 1990 flood caused extensive damage to the north bank levee downstream of State Route 203, primarily along the top of the levee.

Upper South Fork Snoqualmie River

Significant flow events occurred during the month of November in 1986 and 1990, affecting some residential areas along this fast flowing river and causing surface erosion damages to several flood protection facilities (i.e., revetments and levees). The levees are located in the lower portion of the reach with revetments extending farther upstream

2.3.7 Revision 7

Recent large floods at the Carnation gage include November 1990, which had a discharge of 65,200 cubic feet per second and a flood elevation of 60.70 feet.

Two people were killed by flooding in the lower Snoqualmie River basin in the 1990s. Both failed in attempts to drive across the mile-wide valley bottom on flooded roadways. The November 1990 flood killed hundreds of dairy cows and other livestock in the lower Snoqualmie basin. Subsequent floods have not had similar animal mortality, in part because the dairy industry no longer dominates valley land use.

Homes and other structures throughout the lower Snoqualmie basin are subject to flood damage. For the most part, these structures were developed for agricultural use and have been placed on the highest portions of large floodplain parcels. Nonetheless, deep and fast flows are a hazard throughout the lower Snoqualmie River floodplain.

2.3.8 Revision 8

Cedar River – **Within the Area of City of Renton -** Significant flooding problems have occurred along the downstream end of the study reach at the Renton Airport and on Boeing company property. Flooding has also been a problem within the Maplewood subdivision upstream near the Highway 169 Bridge. Periodic surveys of the river channel indicate that the bed of the river in the channelized downtown reach near the airport aggrades fairly rapidly. Significant debris accumulations have also occurred in this reach during large floods. Both factors contributed to substantial flooding at the airport and Boeing during major floods in December 1975 and November 1990, and during a smaller flood in November 1995. Portions of the Maplewood area experienced flooding during these events as well.

The recurring flooding problems at the airport and on Boeing property prompted the development of a USACE Section 205 comprehensive flood-control plan for the lower Cedar River to prevent flooding up to the 1-percent-annual-chance flood. The project included the construction of levees and floodwalls, dredging of the riverbed and modifications to the south Boeing Bridge. USACE constructed the project in 1999 and 2000, which includes a series of levees and floodwalls along both sides of the river downstream of Logan Avenue and a levee on the left bank just upstream of Logan Avenue. The USACE also dredged the riverbed from Lake Washington upstream to approximately Logan Avenue. In addition, the south Boeing Bridge was modified and fitted with hydraulic jacks to lift it above the water during flood events. The levees and floodwalls are USACE certified and designed to provide protection for the 1-percent-annual-chance flood with greater than 90 percent reliability, assuming the channel is periodically dredged as outlined in the Cedar River at Renton

Flood Damage Reduction Project Operation and Maintenance Manual (Reference 144).

Green River - Because of the flow regulation provided at Howard A. Hanson Dam, the Green River does not generally cause significant flooding in the study area. An exception to this is the area at the confluence of Mill Creek/Mullen Slough with the Green River. This area generally floods whenever flows in the Green River at Auburn exceed about 10,000 cfs. As flows approach 12,000 cfs, the flooding in this area may close roads, inundate houses and businesses, and cause other problems. This flooding can have significant impacts, as many people use these roads to access their homes or workplaces. While flooding in the Lower Green River and Middle Green River is likely significantly less pronounced than in the period prior to the construction of Howard A. Hanson Dam (1961), the potential impacts of flooding are now much greater due to the nearly complete development of the floodplain. Factors contributing to flooding along the Lower Green River include channel changes (both natural and man-made) such as aggradations, levees, and revetments.

There is significant overbank inundation along the entire study reach of the Middle Green River during large floods. This flooding, although natural for a low gradient stream, has significant impact to residents along the river, many of whom need to use the roads in the floodplain to access their homes. Flood levels in the lower end of the Middle Green River study reach may also be affected by backwater effects from a large natural log jam near RM 32.3.

Kelsey Creek - Anecdotal information provided by City of Bellevue staff indicates that significant overbank flooding along the Kelsey Creek study reach is minimal, with the exception of flooding near the Illahee Apartments on Bel-Red Road and in the lower reaches near the Glendale Golf Course and within Kelsey Creek Park. Frequent flooding has also been observed on the West Tributary within the golf course and park.

Patterson Creek - There is significant overbank flooding along the mainstem of Patterson Creek during large storms. This flooding, although natural for a low gradient stream, has significant impact to residents along the stream, many of whom cross the floodplain to access their homes. Contributing factors to mainstem flooding include: beaver dams, reed canary grass choking reducing channel capacity and limiting the ability of the channel to cut new channels around barriers such as beaver dams, and backwater effects from the Snoqualmie River. Flood levels in the lower two miles of Patterson Creek may be exceeded by up to 10 feet or more by flood levels on the Snoqualmie River.

Lower Snoqualmie River - The Snoqualmie Valley is a wide, low gradient floodplain mostly comprised of agricultural lands with a few relatively small residential communities. Flooding is most commonly associated with inundation of farm houses and barns, and the valley roads that parallel or cross the mainstem Snoqualmie River. Damage is often due to large areas of inundation along with localized erosion of outer river banks and revetments, overtopping of flood protection levees, and road embankments.

Although the mainstem Snoqualmie is characterized by relatively low velocities and a mild gradient, flooding can cause substantial localized erosion. Problems generally relate to constrictions, where flow energies become concentrated. The Carnation Farm Road is such an example; the road fill embankment forces flood waters through two small bridge openings. Both bridge approaches were washed out during the Thanksgiving 1990 flood event when flood flows exceeded their capacity. Existing King County flood control facilities (levees and revetments) in this basin sustained damages of just over one million dollars in the Thanksgiving 1990 flood.

In addition to this erosion damage, the deep, broad flooding of the Snoqualmie River valley brings other damages. The Thanksgiving 1990 flood killed hundreds of cows on the lower valley's dairy farms. Rising flood waters damaged homes near Carnation and scattered locations elsewhere throughout the valley. In two separate incidents (January 1990 and November 1990), motorists drowned when they attempted to drive across flooded valley roads.

Springbrook Creek – No additional information provided.

2.3.9 Revision 9

Puget Sound

No additional information to add.

Sammamish River - There are no known recent reports, from the County or from local communities, of the Sammamish River flooding into its overbanks. In January 1997, a flow of approximately 2,900 cfs occurred. This is the largest recorded flow at the Northeast 116th Street stream gage since monitoring began in water year 1966. Observed flooding in the overbanks for this event was attributed mostly to tributary flow and local runoff and not direct flooding from the Sammamish River. However, overbank flooding may occur for events approaching the 1-percent-annual-chance exceedance event, especially if water were to escape the berms along the edge of the main channel. Overbank flooding in the middle river segment, where land use is dominated by agriculture, has minimal impact on health and safety, but in more heavily developed areas it can have significant impacts on residents and commercial and industrial properties along the river. Potential impacts of the flooding will continue to increase if development in the floodplain increases. The existence and impact of future flooding problems along the study reach may also be affected by channel changes, constrictions at bridges, and other development in the floodplain.

White River – During large floods there is the potential for significant overbank inundation throughout White River. This flooding, although naturally occurring, can have significant impacts on residents along the river. While flooding along the White River is likely less frequent and less severe than in the period prior to the completion of Mud Mountain Dam in 1948 (Reference 186), the potential impacts of flooding continue to increase as development in the floodplain increases. Flood problems may also be exacerbated by channel changes, constrictions at bridges, and other infrastructure in the floodplain.

2.4 Flood Protection Measures

The Seattle office of the National Weather Service maintains and collects hydrometeorological reports from a network of substations and uses this information to prepare flood forecasts for King County streams. Flood warnings are issued by them and given wide dissemination through all media by cooperative efforts with local and Federal agencies.

Levees exist in the study area that provide the county with some degree of protection against flooding. However, it has been ascertained that some of these levees may not protect the community from rare events such as the 1-percent-annual-chance flood. The criteria used to evaluate protection against the 1-percent-annual-chance flood are 1) adequate design, including freeboard, 2) structural stability, and 3) proper operation and maintenance. Levees that do not protect against the 1-pecent-annual-chance flood are not considered in the hydraulic analysis of the 1-percent-annual-chance floodplain.

Levees on the Tolt River, near its confluence with the Snoqualmie River, provide moderate protection to urban development in the City of Carnation and to adjacent agricultural lands. A 600-acre agricultural area on the left bank of the Snoqualmie River, 1 mile downstream from Fall City, is protected from minor spring floods by a levee approximately 1 mile long. Levees along the lower 2 miles of both banks of the Raging River and its confluence with the Snoqualmie River protect a portion of Fall City and agricultural lands. Levees along the South Fork of the Snoqualmie River provide approximately 2-percent-annual-chance flood protection to the City of North Bend.

Bank erosion occurs at nearly all river stages, but is most severe during medium and high flows. Bank protection projects have been constructed at numerous locations along the Snoqualmie River and its major tributaries by riparian owners, local governmental agencies, and the Federal government.

In 1960, the City of Seattle constructed a water supply project on the South Fork of the Tolt River. Total storage capacity of the reservoir is about 58,000 acre-feet. Although flood control storage is not a project feature, some minor storage of flood discharges does occur.

The USACE operates the Howard A. Hanson Dam at Eagle Gorge on the upper Green River. Completed in 1962, the dam provides approximately a 500- to 600-year level of protection against overbank flooding by the Green River. The dam is a rockfill embankment approximately 235 feet high with a gated spillway and a maximum reservoir elevation of 1,222 feet. Stored water is released as soon as possible after a flood to provide for the possibility of a second flood. The USACE current operation of Hanson Dam provides that all runoff is passed through the dam until the flow at the Auburn gage is expected to reach 12,000 cfs. At that point, further releases are regulated to maintain no more than 12,000 cfs at Auburn.

Channelization and levee construction, primarily downstream of Auburn, has provided additional flood protection for the overbanks. A total of approximately 16 miles of levees have been constructed in addition to roadway systems that function as levees, between State Route 18 at Auburn and the Black River confluence at Tukwila.

Based on information received from the USACE, the levee system along the left (west) bank of the Green River, from Strander Boulevard to RM 16.7, in the City of Tukwila, will adequately provide protection against overtopping or failure caused by the 1-percent-annual-chance flood, with at least 2 feet of freeboard.

King County and the various incorporated cities along the Green River (Tukwila, Renton, Kent, and Auburn) are responsible for maintenance of portions of those levee systems. Since the adoption of enabling legislation by the State of Washington in 1945, the State and King County have combined to control riverbank erosion.

The Black River basin, including Springbrook Creek, has been the object of the ongoing East Side Green River Watershed Study (Reference 30). That study was initiated in 1965 by the former SCS with the support of King County and the Green River Valley cities. The P1 pumping station and storage pond, as part of the plan, were constructed in 1972 and 1984, respectively. A major box culvert replacement was installed at SW Grady Way in 1986 and is considered in this study in its partially obstructed condition. Preliminary plans exist for the construction of the P1 channel from SW Grady Way north to the storage pond and additional culvert replacements under Interstate 405 and SW 16th Street. The timing and funding for construction of these improvements is not finalized; therefore, they are not considered in this study.

A regional detention basin was constructed on Mill Creek (Kent) in 1981 at Earthworks Park in order to provide flood-control storage for reduction in downstream peak runoff. A second smaller upstream detention basin was previously constructed in the Upper Mill Creek basin to provide for additional reduction in peak flows to the lower valley areas. This has reduced the magnitude and frequency of, but not eliminated, flooding problems downstream of the Earthworks Park structure. The City of Kent is developing a plan to construct more detention storage in order to further alleviate their flooding problems.

Partial reduction in peak runoff conveyed to Mill Creek (Auburn) is provided by stormwater detention storage basins constructed on the south tributary to Mill Creek, above its confluence with the Peasely Canyon tributary. Locally referred to as the “Auburn 400” ponds, and located east of and adjacent to State Route 167 and 15th Street SW, they provide an unidentified effect on routing of peak tributary flows to Mill Creek. Additional regional detention storage is being considered for other study reaches of Mill Creek, downstream of State Route 18, in an attempt to maintain adequate storage capacities for limiting downstream discharges with continued floodplain development.

On the White River, peak flows are regulated by the Mud Mountain Dam, a structure built by the USACE. Storage was initiated in 1942, and the project was finally completed in 1953. The structure is an earth and rockfill dam, 425 feet above bedrock. The reservoir has a storage capacity of 106,000 acre-feet of water and is capable of controlling floods 50 percent greater than the maximum flow of record.

Levees have also been constructed along portions of the White River along its course through the Cities of Auburn and Pacific.

The amount of storage provided naturally by Lake Sammamish has a moderating influence on flow, and the channelization project of the Sammamish River by the USACE has significantly reduced flood problems. Major drainage improvement and partial flood protection are provided by the channel improvement project completed in 1966 by the USACE for King County. The project extends from below Lake Sammamish to Kenmore, a distance of approximately 14 miles. The river channel was deepened an average of 5 feet and increased in width from a former average of about 15 feet to the improved 32 to 50 feet. Excavated material from the channel enlargement was used to construct the levees. A low weir with a crest elevation of 28.1 feet NAVD 1988 was constructed to control the outlet of Lake Sammamish. The channel improvement and outlet project provide protection against spring floods with a recurrence interval of 10 years without causing Lake Sammamish to rise higher than elevation 32.6 feet NAVD 1988. No significant flood-control measures have been developed on the Sammamish River tributaries except for channelization of the lower end of Bear Creek at Redmond (Reference 28).

Most of the channel of May Creek is in its natural condition. The lower 1,000 feet have been channelized to alleviate flooding problems caused by channel aggradation resulting from excessive siltation problems.

King County has established a flood fighting plan that is activated when the Cedar River reaches a discharge of about 4,000 cfs at Renton. The plan consists of patrolling and making emergency repairs to contain this discharge. When the flow exceeds 4,000 cfs, efforts are concentrated on protecting the safety of the affected residents and their personal property. The Sheriff's office, the Office of Civil Defense, fire districts, and the Red Cross are notified for assistance.

The lower 1 mile reach of the Cedar River channel was initially stabilized in 1912. King County and the City of Renton have provided major capital improvements and maintenance for flood and erosion control along the Cedar River. This has included riprap bank protection works, bulkheads construction, cleaning, and snag removal. Major reconstruction of levees and bank protection work was accomplished after the December 1975 flood. River channel dredging upstream from the mouth of the Cedar River has been performed, most recently in 1972, in an attempt to maintain 1-percent-annual-chance flood protection of the improved channel system through the City of Renton.

The major flood-control improvement to Thornton Creek is the diversion works with a 72-inch overflow pipeline to Lake Washington. The diversion reduced the peak flows to the lower mainstem reach of Thornton Creek such that only minimal downstream flooding hazards exist up to a 1-percent-annual-chance frequency existing conditions flooding event. This assumes that full unobstructed capacity is maintained to the diversion pipeline.

In 1946, the USACE constructed a levee along the south bank of South Fork Skykomish River in Skykomish. This levee is approximately 970 feet long and provides variable flood protection to a portion of the town.

A flood-protection structure that significantly influences flooding on Des Moines Creek is the road embankment from Marine View Drive located in the City of Des Moines, which creates enough detention storage to reduce the peak 1-percent-annual-chance flood by almost 50 percent on Des Moines Creek.

In 1983, the City of Seattle constructed a regional stormwater detention basin on Longfellow Creek south of SW Webster Street. The detention basin has helped reduce downstream flooding problems, although basin overflow for more severe storms, as evidenced in the January 1986 event, will reduce the basin's effectiveness on reduction in peak flows.

There are no other flood-control measures for other streams studied that significantly reduce flooding.

2.4.1 Revision 1 - Miller Creek

In October 1992, King County completed the construction of the Lake Reba Regional Stormwater Detention Pond, which will attenuate flood flows in Miller Creek. The facility is located at the site of Lake Reba, just south of State Route 518. The effect of this facility has been accounted for in the hydrologic and hydraulic analyses. There are no other major structural flood-protection measures planned for Miller Creek.

2.4.2 Revision 2

Snoqualmie River

No additional information to add.

2.4.3 Revision 3

Raging River

No additional information to add.

2.4.4 Revision 4

North Fork Issaquah Creek - There are no existing flood-protection measures along North Fork Issaquah Creek.

2.4.5 Revision 5

North Creek (LOMR) - The flood-protection system comprises interconnected levees located along three separate project areas: the downstream reach of the levee system for the Quadrant Business Park project area is located along the east bank of North Creek from I-405 to 195th Street Northeast; the levee system for the Koll Business Center project area is located along the east and west banks of North Creek from 195th Street Northeast to Northeast 205th Street; and the upstream reach of the levee system for the Quadrant Monte Villa Center project area is located along the east bank of North Creek from Northeast 205th Street to Monte Villa Parkway.

2.4.6 Revision 6

Tolt River and Upper South Fork Snoqualmie River

No additional information available in Revision 6.

2.4.7 Revision 7

Snoqualmie River and Issaquah Creek

No additional information available in Revision 7.

2.4.8 Revision 8

Cedar River - One additional flood protection measure was identified along the river. This feature, a floodwall constructed by the City of Renton in 2000 to protect the old city hall, extends along the left riverbank just upstream of the library. The floodwall is not certified to FEMA standards, and was consequently disregarded for this flood study. No other flood protection measures exist within the study reach.

Kelsey Creek - The only flood protection measures along the study reach occur along the main stem of Kelsey Creek in Kelsey Creek Park. Here, along the right bank, a 500-foot long earthen embankment approximately 7 feet in height was constructed to keep overtopping flows within Kelsey Creek from entering a low-lying swale area immediately to the west. At one time, Kelsey Creek was actually located in this swale reach, but was later shifted east to its current alignment. Because this embankment fails to meet FEMA's certification requirements, it was not considered to provide flood protection, and was effectively not included in the analysis.

Patterson Creek - No major structural flood protection measures exist or are planned along Patterson Creek.

Green River – The Lower Green River is confined within natural banks or constructed levees over almost its entire length downstream of State Route 18. The King County facility inventory includes approximately 30 separately-named levees or revetments within the study area. Of these, however, only the left bank levee in the Southcenter area (RM 11 to RM 17) is a USACE certified flood protection levee. This certified levee is commonly referred to as the Tukwila 205 levee. The remaining levees are not certified and the hydraulic analyses described herein consider the effects on flood inundations of both "with levee" and "without levee" scenarios.

Interior drainage for much of the Green River floodplain is accomplished via pump stations. The three most significant pump stations in the valley are the Black River (PI) Pump Station (BRPS), the Southcenter (P 17) Pump Station, and the Horseshoe Acres Pump Station. The first two of these pump stations are operated by King County and have design capacities of about 2,900 cfs and 100 cfs respectively. The Horseshoe Acres pump station is operated by the City of Kent and has a discharge capacity of approximately 67 cfs. Numerous smaller pump stations also drain the areas behind the Lower Green River levees. These include the Strander Boulevard, South 180th Street, Washington Avenue, Union Pacific, and South 3rd Avenue Pump Stations.

The only significant structural flood protection measure along the Middle Green study reach is a levee along the left bank of the river between RM 34.5 and RM 35.0, near the SE Green Valley Road Bridge. This levee is not certified and the hydraulic analyses described herein consider the effects on flood profiles of both "with levee" and "without levee" scenarios.

2.4.9 Revision 9

Puget Sound

No additional information to add.

Sammamish River - The most significant structural flood protection measures along the Sammamish River are the in-channel weir at Marymoor Park and berms located along the river banks, principally between Leary Way, near the City of Redmond, and Northeast 145th Street, near the City of Woodinville. These berms act like levees, containing water in the main channel and preventing it from reaching overbank areas. However, these berms are not certified levees and have never been accredited by FEMA. Further discussion of the modeling approach taken for these berms is found in Section 3.2.9 Hydraulic Analysis.

White River- The only significant structural flood protection measures within the studied reach of White River are two small, privately built levees along the right bank of the river, near RM 26.4 and the other near RM 24.0. Neither of these levees is certified and hydraulic analyses indicate that both would be overtopped by the base flood. Therefore, it was not necessary to run "with" and "without" levee simulations as the base flood mapping in the area landward of these levees would be the same in either case.

3.0 ENGINEERING METHODS

For the flooding sources studied by detailed methods in the community, standard hydrologic and hydraulic study methods were used to determine the flood-hazard data required for this study. Flood events of a magnitude that is expected to be equaled or exceeded once on the average during any 10-, 50-, 100-, or 500-year period (recurrence interval) have been selected as having special significance for floodplain management and for flood insurance rates. These events, commonly termed the 10-, 50-, 100-, and 500-year floods, have a 10-, 2-, 1-, and 0.2-percent chance, respectively, of being equaled or exceeded during any year. Although the recurrence interval represents the long-term, average period between floods of a specific magnitude, rare floods could occur at short intervals or even within the same year. The risk of experiencing a rare flood increases when periods greater than 1 year are considered. For example, the risk of having a flood that equals or exceeds the 1-percent-annual-chance flood in any 2-percent annual chance period is approximately 40 percent (4 in 10); for any 90-year period, the risk increases to approximately 60 percent (6 in 10). The analyses reported herein reflect flooding potentials based on conditions existing in the community at the time of completion of this study. Maps and flood elevations will be amended periodically to reflect future changes.

3.1 Hydrologic Analyses

Hydrologic analyses were carried out to establish peak discharge-frequency relationships for each flooding source studied by detailed methods affecting the community.

For those flooding sources being restudied or that are extensions of previous detailed riverine studies, peak discharge results presented in the previous Flood Insurance Studies for King County and the Cities of Auburn, Kent, and Renton (References 1, 2, 8, and 15) were compared with updated discharges estimated to determine the appropriate values to be used in this revised study. The peak discharge estimates assume that existing basin hydraulic structures remain unobstructed and existing upstream dams or impoundment structures remain intact with no changes in operating characteristics.

Discharge-frequency for the Snoqualmie River, South, Middle, and North Forks Snoqualmie River, Sammamish River, North Creek, Bear Creek, Evans Creek, Issaquah Creek, North and East Forks Issaquah Creek, Tibbetts Creek, Vasa Creek, Cedar River, Mercer Creek, Right Channel Mercer Creek, Richards Creek, East and West Tributaries Richards Creek, Kelsey Creek, West Tributary Kelsey Creek, East Branch of West Tributary Kelsey Creek, North Branch Mercer Creek, McAleer Creek, Coal Creek, Lyon Creek, Meydenbauer Creek, North Fork Meydenbauer Creek, South Fork Skykomish River, Maloney Creek, and the Tolt River were developed from USGS stream gaging stations on the respective streams by applying the standard log-Pearson Type III methods with the expected probability correction as outlined by the U.S. Water Resources Council (Reference 31).

Discharge-frequency relationships in the revised study for Raging River, Issaquah Creek, Cedar River, Swamp Creek, May Creek, May Creek Tributary, and Big Soos Creek were developed from streamflow records at USGS gages within those watersheds. The gage reference numbers, descriptions, and periods of record (Reference 32) used in the analyses are summarized below. That listing includes additional gages used for correlating and transferring flows between local, hydrologically similar basins or for comparison of results. The Flood Flow Frequency Analysis computer program (Reference 15) was used to determine the discharge-frequency relationships by applying log-Pearson Type III analysis techniques in accordance with methods presented in USGS Bulletin 17B (Reference 33) to the annual peak flow data for each gage site.

Table 1 below shows the gage record data that were used for the original studies only. The studies subsequent to the original studies should have used more updated gage record information for flood flow estimations.

Table 1 - USGS GAGES

Flooding Source	USGS Gage Ref. No.	USGS Gage Description	Period of Record
Snoqualmie River	12-1490	Near Carnation	1930-1965
	N/A	Near Snoqualmie Falls	1929-1965
	12-1445000	Near Snoqualmie Falls	1959-1978
South Fork Snoqualmie River	N/A	At North Bend	1911-12, 1914-16, 1918-26, 1930-38, 1946-50, 1961-78

Table 1 – USGS GAGES *(Continued)*

Flooding Source	USGS Gage Ref. No.	USGS Gage Description	Period of Record
South Fork Snoqualmie River (continued)			1910-12,1914-18 1920, 1922-26
North Fork Snoqualmie River	N/A	Near North Bend	1930
Middle Fork Snoqualmie River	N/A	Near Tanner	1962-1978
Raging River	12-145500	Near Fall City	1946-1985
Tolt River	12-148500	Near Carnation	1959-1971
Green River	N/A	Near Tukwila	1959-1963
	N/A	Near Auburn	1937-1962
	N/A	Near Black Diamond	1940-1948
Big Soos Creek	12-112600	Above Hatchery, Near Auburn	1961-1986
Sammamish River	1250	At Bothell	1940-1963
	N/A	Near Redmond	1940-1957
Swamp Creek	12-127100	At Kenmore	1964-1986
Little Bear Creek	12-126000	North Creek, Near Bothell	1946-1976, 1986
Bear Creek	12-122500	Near Redmond	1946-49, 1980-81, 1986
	12-124500	At Redmond	1946-50, 1956-58, 1986
	12-124000	Evans Creek, Above Mouth Near Redmond	1956-1977
Issaquah Creek	12-121600	Near Mouth	1964-present
	12-121000	Near Issaquah	1946-1964
May Creek	12-119600	At Mouth, Near Renton	1965-1979
Cedar River	12-119000	At Mouth	1946-1985
	12-1175	Near Landsburg	1948-present
McAleer Creek	12-1276	At Lake Forest Park	1963-72, 1973-74
Mercer Creek	N/A	At Bellevue	1945-present

Flooding Source | USGS Gage Ref. No. | USGS Gage Description | Period of Record

Table 1 – USGS GAGES *(Continued)*

Flooding Source	USGS Gage Ref. No.	USGS Gage Description	Period of Record
Mercer Creek	N/A	At Bellevue	1949-present
South Fork Skykomish River	12-1330	Near Index	69 years
	12-1305	Near Skykomish	26 years
Beckler River	12-1310	Near Skykomish	28 years

Discharge-frequency relationships established for gage locations were transferred to selected runoff concentration points along the study reaches through the application of regional regression techniques per published regression equations (Reference 34).

USGS gage flow records from the adjacent, hydrologically similar North Creek basin were used to establish flow estimates for Little Bear Creek. Evaluation of peak recurrence interval discharges in the lower reaches of Little Bear Creek, downstream of the State Route 202 crossing, include reductions in main channel flow to reflect overflows away from the main channel that enter the Sammamish River at other locations.

Updated hydrology for flooding sources either being restudied or that are extensions of existing detailed studies were compared using statistical confidence limits with existing published Flood Insurance Study discharges at identified locations. Comparison of peak discharge estimates for the May Creek gage site with those published in the previous Flood Insurance Study for the City of Renton indicated no significant statistical differences. Therefore, in accordance with FEMA guidelines, the previous flow estimates for the gage site have been used in the present study.

Recurrence interval peak discharge estimates established for the added detailed study reach of Bear Creek, upstream from its confluence with Cottage Lake Creek, are based on the results of the statistical analysis of annual peak flows at USGS gage No. 12-122500 near Redmond. The limited period of gage record (8 years, including January 1986 event provisional flow estimates) would normally preclude analysis using this method. However, additional gages located on Cottage Lake Creek (No. 12-123000), Evans Creek (No. 12-124000), and on the downstream reach of mainstream Bear Creek, (No. 12-124500) provided adequate data for comparative assessment of results.

Discharge-frequency relationships for Thornton Creek, Longfellow Creek, Mill Creek (Auburn), and Rolling Hills Creek were developed using the USACE HEC-1 computer program (Reference 35). The basic application of this synthetic hydrograph model included computation of drainage subbasin runoff hydrographs using the NRCS Type 1A storm distribution (Mill Creek and Rolling Hills Creek), routing of those hydrographs through channel reaches and detention storage areas, and combining them with downstream subbasin hydrographs at selected study reach runoff concentration points. Calibration of those models to discharge estimates, developed from high water mark data collected for the January 1986 event, was performed.

Peak-flow estimates for Thornton Creek include consideration of unobstructed diversion of flows to the overflow pipeline to Lake Washington. Runoff estimates for Thornton and Longfellow Creek used a multiple peak design storm distribution pattern based on the actual January 17 through 19, 1986, storm event, taken from a network of local precipitation gage data (Reference 36). In addition to localized culvert backwater routing effects, routing of flows through the SW Webster Street detention basin and outlet structure was included in the Longfellow Creek modeling analyses. Discharge estimates computed at the mouth of Mill Creek (Auburn) consider noncoincidence of peak flows in the Green River and Mill Creek. Storage routing effects of backwater storage at the location have therefore not been considered in this analysis. Discharge estimates for Mill Creek for the floodway determination were modified to reflect reduction in storage with encroachment on the storage provided by the natural floodplain.

Recent modeling analyses of the Mill Creek (Kent) and the Springbrook Creek basins using the NRCS TR-20 hydrograph program have been performed by a local consultant for the City of Kent Drainage Master Plan (Reference 37). That study developed 25-year and 1-percent-annual-chance recurrence interval discharge estimates based on a 12-hour duration, NRCS Type 1A storm distribution for the valley floor basins. It included consideration of significant storage-routing components within the Mill/Springbrook Creek system, including the recently completed Earthworks Park stormwater detention facility. The discharge estimates presented in the City of Kent Drainage Master Plan and supplemental computer output files for existing land use conditions have been accepted for use in this restudy. Additional recurrence interval flows were extrapolated from computer flows. The resultant flow estimates were reduced by overflow estimates to Springbrook Creek north of James Street, computed using hydraulic backwater rating methods, to provide the downstream estimates.

Stream gage records are not available for the Black River and Springbrook Creek. In the absence of gaged discharge data for statistical determination of peak flow estimates, information from several previous hydrologic modeling studies within the Black River basin were collected and reviewed for comparison of results and for determination of acceptability for use in the restudy. Synthetic unit hydrograph modeling of basin runoff has been performed by the USACE using the HEC-1 Flood Hydrograph model (Reference 38); the NRCS using the TR-20 model (Reference 39); by the previous study contractor for the existing Renton Flood Insurance Study using the TR-20 model (Reference 15); and, more recently, by other consultants for upstream reaches of the basin, using the TR-20 model. The flow estimates from the previous Flood Insurance Study were determined to be the most representative of the conditions existing in the basin at the time of this restudy, and were therefore used. Since the previous Flood Insurance Study only calculated the 25- and 100-year hydrographs, the 0.2-percent-annual-chance hydrograph was interpolated from those previously computed. The 0.2-percent-annual-chance hydrograph was not analyzed because of the extensive changes in overbank storage that occur at P1 pond stages in excess on the 1-percent-annual-chance recurrence interval.

The Green River basin has been studied extensively by the Seattle District of the USACE. The USACE operation of Howard A. Hanson Dam provides flow regulation for flood protection to the downstream river reaches, particularly the lower Green River Valley downstream from Auburn. Current USACE operation of the dam regulates the peak downstream flow releases up to the standard project flood to 12,000 cfs at the USGS Auburn gage. This includes consideration of tributary inflows downstream of the dam (i.e., Newaukum Creek and Big Soos Creek). Discharge-frequency analyses have been performed by the USACE as part of the Green River Flood Reduction Study (Reference 23) for estimation of peak unregulated and regulated floodflows at the Auburn stream gage. Results of those analyses were reviewed and used in this restudy. The flows are also in agreement with previously published Flood Insurance Study discharge estimates.

Discharge-frequency relationships for the White River were obtained from a backwater channel-capacity study by the USACE (Reference 40). The selected stations were Mud Mountain Dam and the White River at the mouth. The peak discharges were adjusted for the White River near Auburn. Those adjusted discharges were used directly for this study.

Because there are no streamflow records on Forbes Creek and Yarrow Creek, runoffs for the floods of interest were calculated using rainfall relationships developed for the area and a computerized stormwater routing model. The model incorporates the unit hydrograph methodology developed by the NRCS (Reference 41). The peak discharges obtained by this method were comparable to those derived from regional regression equations published by the USGS (Reference 42).

The hydrologic analysis of Miller and Walker Creek in the Sea-Tac communities plan (Reference 43) used a stormwater management model developed in earlier river basin studies. A single large storm and measurement at temporary gaging stations along the creeks were used to calibrate the model, and flows for the 10-, 2-, and 1-percent-annual-chance storms were computed. The 0.2-percent-annual-chance flood was estimated by extending the curve through the computed points.

A gaging station on Miller Creek was established in 1973 to provide a better understanding of hydrologic conditions in the stream.

No gage records exist for Des Moines Creek. Because of highly similar drainage basin characteristics, peak discharges per square mile for Miller and Walker were applied to the Des Moines Creek drainage basin. These flows gave flood elevations well in excess of local experience. The excessive flow rates were explainable because an 80-foot-high road embankment (Marine View Drive) crosses Des Moines Creek Canyon at the upper end of the detailed study area. The box culvert flowing under the embankment has a capacity of 300 cfs before peak flow storage begins. However, even assuming that no outflow was allowed, the impoundment can store 65 percent of the runoff that would occur during a 6-day, 1-percent-annual-chance storm. Therefore, reservoir routing capacity exists to significantly reduce peak flows. Utilizing rainfall-runoff data and techniques developed by the study contractor during a recent study of a similar urban area located several miles to the north, a 1-percent-annual-chance synthetic runoff hydrograph was developed for a 6-day storm for Des Moines Creek.

The 1-percent-annual-chance hydrograph was routed through the storage reservoir created by the road embankment, reducing the unrouted peak discharge. This same percentage of flow reduction was applied to the 10-, 2-, and 0.2-percent-annual-chance unrouted peak flows.

3.1.1 Revision 1

Miller Creek - Miller Creek passes through several communities as it flows downstream to Puget Sound. The upper end of the study reach passes through the newly formed City of SeaTac. About mid-reach, the channel passes under Des Moines Way (near State Route 509) and enters unincorporated King County. Downstream of 1st Avenue South, near 6th Avenue SW, the channel enters the City of Normandy Park and remains within the city limits until it empties into Puget Sound. Land neighboring the stream channel is occupied by private residences and forest, farm, and pasture lands.

The average annual precipitation, as recorded at the nearby Seattle-Tacoma International Airport, is approximately 38 inches. The heaviest rainfall occurs during the months of November through January, with little rainfall during the summer months of July and August. The average annual temperature is 50°, with average daily high of 59°F and lows of 44°F. July and August are the warmest months, with average daily maximum temperatures of 75°F, while January is the coldest, with average daily minimum temperatures of 34°F.

Estimation of flood discharges along Miller Creek and its tributaries was based on a previous study performed by nhc in 1990 for the King County Division of Surface Water Management (Reference 95). In this study, the Environmental Protection Agency’s Hydrologic Simulation Program – FORTRAN (HSPF) model (Reference 96), was used to describe the hydrology of the Miller Creek basin. HSPF is a state-of-the-art hydrologic simulation model that is rapidly becoming the model of choice for simulating streamflow values by many government and private agencies. The model was used to compute time series of streamflows estimated from observed rainfall, evaporation, and soil-characteristic data. The model included the effect of the Lake Reba Regional Stormwater Detention Pond, which was constructed in 1992 near the headwater of Miller Creek.

The Miller Creek basin HSPF model was calibrated using 2 years of recorded streamflow data collected at a gage near the Southwest Suburban Sewer District treatment plant, recorded precipitation at the National Weather Service SeaTac weather station, and evaporation data from the Puyallup station. Calibration was performed for current basin land-use conditions.

To develop flood-frequency curves, the calibrated model was then used to stimulate Miller Creek streamflows. A time series of streamflow values was created for the 29 years between October 1, 1961, and January 11, 1990, using historical SeaTac precipitation and Puyallup evaporation data. Log-Pearson III distributions were fit to the annual peaks from the simulation to determine the 10-, 2-, 1-, and 0.2-percent-annual-chance flood discharges for Miller Creek. It should be noted that considerable extrapolation was required to determine the 1-, and 0.2-percent-annual-chance flow rates. The areas of Tub Lake Tributary make up part of the total of 22 subbasins of the main stem of Miller Creek. Flood estimates for the Tub Lake Tributary were also computed using the HSPF model.

3.1.2 Revision 2

Snoqualmie River - No additional information to add.

3.1.3 Revision 3

Raging River - The discharge values for the downstream reach were developed using a statistical analysis of the stream-gage data at USGS Gage No. 12145500 along the Raging River. The period of record from 1945 to 1992, plus an historic event in 1932, was used in the analysis. The discharge values from this revised hydrologic analysis are significantly higher than the discharge values from the Summary of Discharges Table in the previous Flood Insurance Study for King County, Washington and Incorporated Areas, dated September 29, 1989 (Reference 94), which were used in the detailed study performed by CH2M HILL, Inc., for the reach upstream of I-90. Therefore, FEMA revised the discharge values for the upstream reach using drainage area-discharge relationships established in the detailed hydrologic analysis for downstream reach.

3.1.4 Revision 4

North Fork Issaquah Creek - North Fork Issaquah Creek originates in King County just northeast of the City of Issaquah and flows in a mostly southwesterly direction to the main stem of Issaquah Creek. The contributing basin area is 4.5 square miles, ranging in elevation from approximately 60 feet near the mouth to a maximum elevation of approximately 1,200 feet.

Much of the upper basin was forested as of 1989. Since then, the major “Klahanie” urban development has largely been completed and covers most of the northern side of the upper basin. A second major urban development, “Grand Ridge,” is presently in the planning stages and will cover most of the southern side of the upper basin.

Flows estimated using the HSPF model are based on a model that was calibrated to streamflow data collected on North Fork Issaquah Creek for the years 1988 through 1990, based on the forested land-use conditions that existed. Peak flows from the calibrated HSPF model are substantially lower than other estimates primarily because the basin contains proportionally more highly permeable outwash soils than other gaged basins in the regions.

Revisions to the King County HSPF model were made as part of the restudy to reflect major residential developments that have been constructed since 1989 and others that were in the planning stage as of 1995. The Klahanie and Grand Ridge developments will cover essentially all of the upper basin area. Both of these developments are located primarily within the North Fork Issaquah Creek basin, but extend across basin boundaries into other basins as well.

The Klahanie project is an 856-acre development located in the upper North Fork basin north of the Issaquah Fall City Road and covering approximately 25 percent of the North Fork basin (Reference 101). Construction for this development began in 1987 and was nearly complete as of 1995. Stormwater peak flows are controlled through a series of detention ponds including a major facility developed by construction of a control structure at the outlet of Yellow Lake within the development area. The stormwater facilities for the Klahanie development, and the Yellow Lake outlet control in particular, were designed so that peak flows leaving the site would not be increased as a result of the development.

The Grand Ridge project is a proposed 2,200-acre development located in the upper North Fork basin south of the Issaquah Fall City Road and which will cover approximately 50 percent of the North Fork basin. Environmental Impact Statement hearings for this project were in progress during 1995. Discussions with the project's engineers revealed that stormwater control is planned to be provided entirely through infiltration systems, which will preclude peak flows from developed areas being released directly to the stream system. With infiltration systems, the Grand Ridge development is not expected to cause any significant increase in peak flows in the North Fork basin.

While updating the HSPF model, it was discovered that the major stormwater detention control facility at Yellow Lake had been constructed in 1987 in advance of most other Klahanie development activity, but had not been included in the original HSPF model. Calibration of the original model had been attained to some extent by adjusting the model's previous surface runoff parameters to reflect the flow attenuation effects actually caused by the outlet controls at Yellow Lake (Reference 102).

Because of the changing land use, neither the original calibrated HSPF model nor the revised model with 1989 land use are directly suitable for estimating flood discharges for 1995 conditions. The original flood frequency curve for the calibration period is artificially suppressed because of the timing of the HSPF calibration in relation to phasing of the Klahanie development: the Yellow Lake stormwater facility had been constructed, but the development to be serviced by that facility had not. The flood frequency curve from the revised model with 1989 land use underestimates the calibration-period flows by about 25 percent.

For purposes of the restudy, it is assumed that flows from the "HSPF Model Revised, 1995 Land Use" underestimate actual flows by 25 percent. The 25-percent value is based on the peak-flow reduction that resulted when the original calibrated model based on the 1989 land use was revised to include the Yellow Lake outlet control. For the restudy, a 1-percent-annual-chance discharge of 315 cfs was used near the mouth of North Fork Issaquah Creek.

Flood discharges in the lower portion of the restudy reach are supplemented by floodwater originating from the main stem Issaquah Creek. Main stem Issaquah Creek channel overtopping between the I-90 crossing and the confluence with the North Fork channel is shown by high-water-mark information to have occurred during the November 1986 flood, and probably also the January 1990 flood, which had a nearly identical main-channel discharge (Reference 103). These floods each have a return interval of approximately 30 years. Water that overtops the right bank of the main stem Issaquah Creek channel downstream of the I-90 crossing will flow toward the North Fork channel.

Bear Creek - Most of the Bear Creek study reach lies along Avondale Road NE, which is the extension of State Route 520. Avondale Road NE runs primarily north-south and crosses Bear Creek at three locations approximately at River Miles 1.4, 5.4, and 5.7 (Reference 107). The most upstream Avondale Road NE crossing is the upstream limit of the restudy. Bear Creek originates in an extensive network of wetlands near Paradise and Echo Lakes in southern Snohomish County, and flows primarily southward for approximately 14 miles to its confluence with the Sammamish River (Reference 105). The contributing drainage area is approximately 51 square miles at its mouth.

The lower portion of the restudy reach flows through a flat floodplain that ranges in width from approximately 250 feet wide downstream of Union Hill Road to nearly 1,800 feet wide downstream of the confluence of Bear and Evans Creeks. Most of the lower portion of the floodplain is bounded by road or business park fills including those of State Routes 202 and 520, Union Hill Road, Avondale Road Extension, Avondale Road NE, Bear Creek

Business Park (Harvard College), and Redmond Village. Beginning approximately 0.5 miles upstream of the confluence with Evans Creek, the Bear Creek floodplain is generally narrower, ranging between 200 and 350 feet wide, bordered by gentle, rolling hills.

Some flow may overtop sections of Union Hill Road upstream of Avondale Road NE. Although the area near Union Hill Road is presently developed, these flows were assumed to be relatively minor.

South Fork Skykomish River - The peak discharge-frequency relationships for the reach of the South Fork Skykomish River below the confluence with Beckler River were developed using a statistical analysis of the stream-gage data from the Index, Washington gage (No. 12133000). This gage has a total of 74 records for the water years ranging from 1897 through 1982. The hydrologic analysis for the South Fork Skykomish River upstream of the confluence with Beckler River was based on the annual peak-flow data from the Skykomish gage (No. 12130500), with 26 years of record from water years 1930 through 1970. The floodplain boundaries along the South Fork Skykomish River in Snohomish County are based on an approximate study and do not match those from the detailed study in King County at the county line.

Middle Fork Snoqualmie River - The hydrologic analysis for Middle Fork Snoqualmie River was based on flow rates from the previous effective FEMA study.

North Fork Snoqualmie River - The hydrologic analysis for the North Fork Snoqualmie River was based on peak-flow gage data on the river from the gages near North Bend, Washington (No. 12143000), and Snoqualmie Falls, Washington (No. 12142000). The North Bend gage includes 43 records from 1909 to 1978. The Snoqualmie Falls gage includes 61 peak records from 1930 to 1992.

3.1.5 Revision 5

North Creek - Peak discharge-frequency relationships for the revised reach of North Creek were determined from the hydrologic computer model developed for the original study of North Creek using the U.S. Environmental Protection Agency HSPF model (Reference 119). For the original study, the North Creek HSPF model was run with 39 years of 15-minute rainfall and daily evaporation to develop flood-frequency curves. The resulting 39-year time series of simulated North Creek stream flows were used to create 39 years of annual instantaneous peak flow data at four locations along the study reach. A Log-Pearson Type III distribution was fitted to the annual peaks using the procedures of Water Resources Council Bulletin 17B, and the magnitudes of flows with return periods of 10-, 2-, 1-, and 0.2-percent-annual-chance flows, respectively, were determined.

Two small streams were identified for study by approximate methods. Horse Creek originates in a steep, wooded gully near the northern corporate limits and drains approximately 1 square mile. It flows through downtown Bothell in a series of culverts, ditches, and closed pipes. The unnamed creek that flows north along 96th Avenue Northeast drains approximately 0.6 square miles of wooded area south of the Sammamish River (Reference 122).

3.1.6 Revision 6

Tolt River - The hydrologic analysis for the Tolt River was based on a statistical analysis of peak-flow data from the gage near Carnation, Washington (No. 12148500). This gage has a total of 58 water years of record: 1929, 1931, and 1938 through 1993.

Upper South Fork Snoqualmie River – The hydraulic analysis of Upper South Fork Snoqualmie River upstream of the I-90 bridge was initially performed by Harper Righellis Inc. The data prepared by Harper Righellis were incorporated into analyses performed by the USACE and revised where necessary.

Middle and South Fork Snoqualmie River - Hydrologic analysis records for the various gages on the Snoqualmie River system were intermittent. Missing data in the intermittent records were synthetically reconstituted using the USACE Regional Frequency computer program HEC-REGFRQ (Reference 124). This program fills in and extends the records for all gages using flow data at nearby long-record stations. All stations above the Snoqualmie near Carnation station were included in the initial HEC-REGFRQ analysis. This initial HEC-REGFRQ analysis significantly improved the station statistics (primarily the regression coefficient and equivalent record length) for all stations except the Snoqualmie near Snoqualmie gage. Therefore, this station was eliminated from the analysis and the final HEC-REGFRQ analysis included only the gages on the South Fork. The reconstituted period of record for these gages was 89 years, from approximately 1909 to 1997.

A two-station comparison with the long-term gage at Carnation was used to extend the record for the short-term gage at Snoqualmie.

Log-Pearson Type III frequency curves were computed for all the gages with the USACE Flood Frequency Analysis computer program HEC-FFA (Reference 125) using the reconstituted HEC-REGFRQ data as input for the gages on the North, Middle, and South Forks. The extended record from the two-station comparison was used as input for the gage on the mainstem of the Snoqualmie River near the City of Snoqualmie.

Discharges at locations other than the gages were computed using drainage area ratio equation with the nearest gage.

The resultant frequency curves were compared with previously published discharges in the Flood Insurance Study. With a few minor exceptions, the previously published discharges for the South Fork gages at the City of North Bend fell within the 25 percent and 75 percent confidence limits of the newly computed frequency curves. Therefore, the previously published discharge for the North Bend station was adopted for this restudy.

3.1.7 Revision 7

Snoqualmie River - The hydrologic analyses for this restudy were based on the USACE study completed in December 1998. The hydraulic analyses were performed by Harper Houf Righellis Inc. and completed in October 2001. This restudy effort was identified in Cooperating Technical Community Memorandum of Agreement dated September 26, 2000, between King County and FEMA.

Regulatory floodways were computed for all studied reaches of the Snoqualmie River; however, only the 1-percent-annual-chance flood event was analyzed for Ribary Creek and Gardiner Creek.

Hydrologic analyses were performed to establish peak discharge-frequency relationships for each flooding source affecting the communities that was studied by detailed methods. This hydrologic analysis was completed by Harper Righellis subsequent to Revision 6 (See section 3.1.6.).

The peak flows used in the steady-state analysis for the three forks of the Snoqualmie River were derived from values previously accepted by FEMA, based on the hydrologic analyses performed by the USACE, Seattle District, for South Fork, as described in Section 10.6.

The peak flows for Gardiner Creek and Ribary Creek were not based on runoff from their catchments, both of which are 1.3 square miles, but rather from an overflow of South Fork through an assumed breach in the left levee. At the downstream end, the 1-percent-annual-chance discharge for Ribary Creek used for this restudy is 2,675 cfs, which is the combined South Fork overflow and the Ribary Creek flow. At the upstream end, the combined South Fork overflow and the Ribary Creek peak flow is 2,950 cfs. The Gardiner Creek 1-percent-annual-chance discharge at the downstream end is 575 cfs, which combines Gardiner Creek, South Fork, and the Ribary Creek split flow. The Gardiner Creek split of the combined South Fork overflow and Ribary Creek flow is 275 cfs (Reference 130). Discharges are shown in tabular format in Table 2.

Issaquah Creek - Hydrologic analyses were performed to establish updated recurrence interval peak discharge estimates for Issaquah Creek and East Fork (Reference 134). For those flooding sources being restudied or those that are extensions of previous detailed riverine studies, peak discharge results presented in the previous FIS for King County and in the Issaquah Creek Basin Plan (Reference 135) were compared to updated estimated discharges to determine appropriate values for this revised study. The peak discharge estimates assume that existing basin hydraulic structures remain unobstructed and that existing upstream dams or impoundment structures remain intact, with no changes in operating characteristics.

Discharge-frequency analysis in this revised study for Issaquah Creek and East Fork were performed as described in the hydrologic memorandum completed for this study (Reference 134). The Flood Flow Frequency Analysis computer program HEC-FFA (Reference 136) was used to determine the discharge-frequency relationships by applying log-Pearson Type III analysis techniques, in accordance with methods presented in the USGS publication *Guidelines for Determining Flood Flow Frequency, Bulletin 17B* (Reference 137) to the annual peak flow data for the gage sites.

The resulting flood flow frequency results for the Issaquah Creek gages and reported/adjusted periods of record were compared to previously published flood flow frequency values. In accordance with *Bulletin 17B* guidance, a generalized skew of -0.02 was used as a HEC-FFA input parameter applicable for this region.

Flood flow frequency analyses also were completed for the period 1964-75 in an attempt to validate the published FEMA record. The computed 1-percent-annual-chance peak flow result was much lower (2,990 cfs) than the 1-percent-annual-chance peak flow previously published (4,700 cfs). The expected probability estimate of 3,410 cfs was also considerably lower.

The revised flood flow frequencies were used because the difference compared to the previous flood flow frequencies was statistically significant. The updated flood flow frequency results computed at Gage 12121600 were adopted for the FIS restudy. (The actual record used was for the period 1964-99 with some updates and was based on no loss of flow from Issaquah Creek.)

Flood flow frequency on East Fork could not be analyzed directly because of the limited stream gage record. Therefore, confidence limits could not be computed to measure against the standard FEMA criteria for acceptance of prior or new flood flow estimates. Considering the similarities in peak flow between the King County Basin Plan Modeling Results (for existing conditions) and the flood flows estimated from gage transfer (using USGS gage 12120600), the higher of those two flow estimates was adopted. Additional documentation of the hydrologic analysis procedures and results are found in the hydrologic analysis memorandum (Reference 134).

Discharge-frequency relationships established for gage locations on the creeks were transferred to selected runoff concentration points along the study reaches through the application of standard USGS methods for transfer of peak flow records (Reference 138).

An analysis of streambank overflows was conducted at five locations along Issaquah Creek (Reference 139). On Issaquah Creek, recurrence interval overflows were taken into account to establish peak flow estimates for downstream reaches. Overflows are located at the Pickering reach, two places along the Gilman reach, the Dogwood Street bridge, and the Newport Way bridge. An overflow path upstream of Gilman Boulevard was rated, and a separate overflow model was developed that extends approximately 0.6 miles downstream (northwest) of the main channel.

Two overflow paths were identified on East Fork, one located on the west bank upstream of the Dogwood Street bridge and the Crescent Drive footbridge. The discharges for the stream studied by detailed methods are shown in Table 2, "Summary of Discharges." However, the following estimates account for current loss of flows upstream and downstream of Gilman Boulevard.

3.1.8 Revision 8

Cedar River – City of Renton Area - The City of Renton provided the peak discharge values used herein to nhc. The flow values were developed by King County (King County, March 2000). The 10-, 2-, 1-, and 0.2-percent-annual-chance flows (see Table 2) were based on a flood frequency analysis of approximately 80 years of peak flow data, fit with a Log-Pearson III distribution.

Cedar River – King County Unincorporated Area - The hydrology developed by King County in March 2000 was first used by Harper Righellis for the Unincorporated King County study of the Cedar River and then, subsequently utilized in the LOMR for the City of Renton.

Kelsey Creek - Flood frequency quantiles for current conditions on Kelsey Creek and the West Tributary were estimated using a Hydrologic Simulation Program - Fortran (HSPF) model originally developed by nhc, and later updated by City of Bellevue staff.

Patterson Creek - Hydrologic analyses were conducted out to establish the peak discharge-frequency relationship for Patterson Creek. A Hydrologic Simulation Program FORTRAN (HSPF) rainfall-runoff model was developed, calibrated, and applied to simulate a 57-year record of flows for the basin. Annual peak flows were extracted from the model at seven locations along the study reach and flow quantiles at each location were estimated by fitting flood frequency curves to these data. The estimated 10-, 2-, 1-, and 0.2-percent-annual-chance floods for existing land use conditions are summarized in Table 2.

Snoqualmie River - The objective of the hydrologic analysis in this study was to develop 10-, 2-, 1-, and 0.2-percent-annual-chance (*i.e.* "N"-year) design flood hydrographs for input to the HEC-RAS unsteady hydraulic model at all model inflow points. Design flood hydrographs were developed for eleven inflow locations along the Snoqualmie River portion of the study area. Inflow points include the upstream boundaries of each river, major tributaries, and areas contributing significant direct discharge to the rivers.

Design event inflow hydrographs were developed using a process that included model calibration, application of the model to simulate a wide range of historic flood events, stage frequency analysis on the resultant historic flood stages at key locations, and then refinement of the N-year design event hydrologic inputs to achieve reasonable concurrence with the corresponding N-year stages at the key locations.

Inflow hydrographs from sixteen of the largest flood events that occurred between water years 1966 and 2003 were synthesized for input to the hydraulic model. The primary source of these flow data were USGS observed flow records. Where USGS data was not available, a range of methods were utilized to estimate historical flood hydrographs at the hydraulic model inflow points including gage data transposition, rainfall-runoff modeling, and reservoir operations modeling.

Each of the sixteen historic floods was then simulated using the HEC-RAS unsteady hydraulic model. For water years in which two significant flood events occurred, both were simulated and the highest stage at each key location was retained. The resultant peak stages were then plotted on frequency paper and stage frequency curves were drawn through the data.

Of all of the floods simulated with the hydraulic model, two were found to produce stages that most closely corresponded to certain N-year stages at key locations throughout the study area. Peak stages produced by the December 1977 flood simulations most closely approximated 10-percent-annual-chance stages in the study area while the November 1990 flood simulations resulted in river stages that most closely matched 2- and 1-percent-annual-chance conditions. November 1990 is also the largest flood within the USGS's systematic gage record, and best suited for developing 0.2-percent-annual-chance design hydrographs. Consequently, historical inflow hydrographs for these two historic floods with relatively small adjustments were used to produce the N-year design input hydrographs for floodplain mapping, floodway analysis, and discharge quantile estimation. The resultant discharge quantiles are summarized in Table 2. These data represent the peak flows simulated in the hydraulic model at the listed locations using the corresponding design event model. The listed locations were included in Table 2 because the USGS operates a stream gage at each of the sites, allowing for easy comparison with the often reported discharges. It should be noted that during large flood events, water escapes the main channel at the Carnation and Duvall gages. Therefore, discharge quantiles at these sites is further divided into discharge remaining in the main channel and that passing by the gage in the overbank.

Springbrook Creek - Springbrook Creek drains a basin of approximately 25 square miles located in a highly urbanized area of western King County, Washington. The basin is bounded on the west of the Green River levee system and on the east by the uplands of the Soos Creek basin. The creek drains portions of the cities of Kent, Renton, Tukwila and unincorporated King County; however, Kent to the south and Renton to the north are by far the largest areas within the basin.

Green River – Hydrologic analyses were conducted to establish the peak discharge-frequency relationship for the Lower Green River. Annual peak flows for the post Howard A. Hanson Dam period (1961-2007) were obtained from the USGS for the Green River near Auburn (USGS gage 12113000) and flow quantiles were estimated by fitting flood frequency curves to these data. Because flood flows in the study reach are highly controlled by regulation at Howard A. Hanson Dam (HAHD), the applicability of traditional flood frequency analysis techniques may be questionable. Therefore an alternative, "target flow plus residual" model was also conceptualized and applied to estimate peak discharge quantiles at the Auburn gage. The estimated peak discharge quantiles at the Auburn gage were then transposed to other locations and adjusted to account for flow attenuation in the Mill Creek/Mullen Slough floodplain plus other inflows within the study reach.

In addition to the peak discharge quantiles, discharge hydrographs for the 1-percent-annual-chance event were needed to allow hydrodynamic simulations of the levee failure scenarios. The Green River discharge hydrograph for the upstream end of the study reach was developed by scaling up the rising limb portion of the November 2006 flood event to match the 1-percent-annual-chance peak flow quantile (12,800 cfs) and then appending to this the Corps' 1-percent-annual-chance regulated condition target flow (i.e. 9 days at 12,000 cfs). This hydrograph thus maintains the peak discharge characteristics of the steady state hydrology but also reflects the Corps' theoretical regulated discharges from Howard A. Hanson Dam. An argument could be made that this hydrograph is overly conservative since the flows at Auburn in the 47 years since the construction of HAHD have never been sustained at levels as high as the theoretical condition for more than a few days. However, because the authorized operation at HAHD calls for targeting a flow of 12,000 cfs for 9 days in a 1-percent-annual-chance event, and because these discharges may be required for HAHD to be able to safely accommodate the Standard Project Flood, it was determined that incorporation of the extended duration of high flows together with the instantaneous peaks was appropriate for this study.

Hydrographs at other downstream inflow points were developed by various estimation techniques. For Mill Creek, Mullen Slough, Midway Creek, and Springbrook Creek flow hydrographs were created using the 1-percent-annual-chance, 9-day discharge for those streams based on past hydrologic modeling efforts by nhc**.** Inputs to the Green River at other locations (primarily pump stations and tightlines) were estimated based on the pump station or tightline capacity. It should be noted that the cumulative total of these downstream discharge points is approximately 10 percent of the discharge on the Green River near Auburn and thus any errors introduced by the estimation techniques are not expected to have any significant effect on the hydraulic simulations.

Hydrologic for Middle Green River analyses were conducted to establish the peak discharge-frequency relationship for the Middle Green River. Hydrologic data for the post Howard A. Hanson Dam period (1961-2007) were analyzed to determine the peak flow data for floodplain mapping. Annual peak flows were derived at three locations along the study reach, at the upstream study limit, at the confluence with Newaukum Creek, and at the confluence with Soos Creek, and flow quantiles at each location were estimated by fitting flood frequency curves to these data.

3.1.9 Revision 9

Sammamish River - Defining appropriate hydrologic data for the Sammamish River was key to developing accurate floodplain analysis. The most significant challenge in developing these data was determining how flow hydrographs from the various tributary basins coincide in time to produce a given peak flow quantile on the Sammamish River. Tributaries to Lake Sammamish comprise a little less than half of the Sammamish River basin area upstream of Lake Washington and outflow from the lake significantly attenuated. Consequently, local runoff and tributary inflows downstream of Lake Sammamish are likely to peak much earlier than lake outflows. The downstream sub-basins, namely (Big) Bear Creek, Little Bear Creek, North Creek, and Swamp Creek, may also peak at different times from each other due to differences in precipitation patterns, land-use conditions (including level of urbanization) and basin storage characteristics. The hydrologic analysis specifically accounted for these differences; otherwise Sammamish River flows could be overestimated and result in an overly conservative floodplain analysis.

To address these timing issues, NHC generated long-time series of flows at multiple points along the Sammamish River (including the weir) under current watershed conditions. These time series were generated using a combination of two models; Hydrologic Simulation Program-FORTRAN (HSPF), to stimulate flow inputs from the tributary basins, and HEC-RAS, to route these inflows down the Sammamish River. Frequency analysis was then performed on the simulated peak flows to determine flows for use in the steady state HEC-RAS model for floodplain analysis. The use of continuous hydrologic modeling (HSPF) precluded the need to make judgments regarding the temporal correlation between tributary hydrographs and Sammamish River peak flows. It also allowed NHC to define flood frequency quantiles based on simulation of actual hydrologic response (with the most recently available land use and weir conditions).

HSPF Modeling

NHC obtained and used existing King County and Snohomish County HSPF models of basins tributary to Lake Sammamish and to the Sammamish River to produce a 60-year time series of flows for each basin (water year 1949 to 2009). The most recent available model for each of the tributary basins was used. King County developed and calibrated HSPF models for all basins in the Sammamish River watershed as part of its Sammamish-Washington Analysis and Modeling Program (SWAMP). These models included Issaquah and Tibbetts Creeks, East Lake Sammamish Tributaries, West Lake Sammamish Tributaries, Bear Creek (including Evans Creek), Little Bear Creek, North Creek, Swamp Creek, and local drainage to the Sammamish River. Land-use conditions represented in these models were from 1995. In more recent work for Snohomish County, NHC has developed and/or updated and calibrated HSPF models for the Swamp, North and Little Bear Creek basins. These models use land-use data current to the time of this flood study analysis (2004 to 2008).

The initial simulation periods for the models provided by King County were limited to the period of record of the local precipitation gages used as input (approximately water years 1990 through 2003). To produce long-term simulations, precipitation records were extended back to October 1948 by transposing SeaTac precipitation data to each of the local gage locations. A multiplier on SeaTac precipitation was determined for each gage using the ration of local to SeaTac mean annual precipitation from overlapping periods of record. Local gage precipitation records used in the provided Snohomish County models had been extended by similar methods. NHC also extended the simulation period forward to 2009 to capture some recent large storm events. As needed, data gaps were filled by transposing data from nearby gages.

Unsteady HEC-RAS Modeling

NHC used the HSPF-simulated time series from each tributary as input to an unsteady HEC-RAS model (development of the HEC-RAS model geometry and model calibration is discussed in Section 3.2.9). The unsteady HEC-RAS model was used to route flows through Lake Sammamish and down the Sammamish River. An unsteady RAS model is preferable to HSPF for the river routing for several reasons:

- The HEC-RAS model geometry was already being developed for this study, and Lake Sammamish and the Sammamish River are not included in the existing HSPF models.
- HEC-RAS can directly model backwater effects and other dynamic conditions on the Sammamish River and at the weir, while HSPF requires static stage-discharge rating curves.

Lake Sammamish was modeled as a "storage area" in the unsteady HEC-RAS model with summed inflows from Issaquah, Creek, Tibbetts Creek, and the East and West Lake Sammamish tributaries, as well as precipitation gains and evaporation losses on the lake surface. Precipitation and evaporation fluxes to the lake surface were generated from data sets used in the HSPF modeling assuming a constant lake surface area. HSPFsimulated flows from Bear, Little Bear, North and Swamp Creeks were modeled as lateral inflows to the Sammamish River. Other local runoff to the Sammamish River was modeled as uniform lateral inflows in five separate reaches: Lake Sammamish to Bear Creek, Bear Creek to Little Bear Creek, Little Bear Creek to North Creek, North Creek to Swamp Creek, and Swamp Creek to the mouth. For the 60-year simulation, the weir was assumed to always be in place and the HSPF land uses unchanging (e.g. not reverting back to 1950s land use) to determine flow quantiles for the watershed conditions that exist today.

Flow Frequency Analysis

NHC performed flow frequency analysis on the 60 years of simulated Sammamish River flows to identify 10-, 2-, 1- and 0.2-percent-annual-chance exceedance flow quantiles at key locations along the Sammamish River between Lake Sammamish and Lake Washington. Flow quantiles were determined at the upstream end of the Sammamish River, at major tributary confluences (i.e, Bear, Little Bear, North, and Swamp Creeks), near Northeast 116th and Northeast 145th Streets, and at the confluence with Lake Washington, the computed peak flows were then used as input to steady HEC-RAS simulations to evaluate the 10-, 2-, 1- and 0.2-percent-annual-chance exceedance water surface profiles and define the regulatory floodway.

White River – Hydrologic analyses were conducted to establish the peak discharge-frequency relationship for the White River. Hydrologic data for the post Mud Mountain Dam period (1946-2007) were analyzed to determine the peak flow quantiles for use in the floodplain mapping. Annual peak flow quantiles were estimated at three locations along the studied reach: approximately 1,275 feet downstream of State Highway 410; at the confluence of Red Creek; and at the confluence of Boise Creek. Flow quantiles approximately 1,275 feet downstream of State Highway 410 were estimated using frequency analysis of the adjusted dam discharge record as described by NHC. Quantiles for the intervening locations were estimated based on the proportionate share of the local basin tributary to the White River between Mud Mountain Dam and the analysis point.

Peak discharge-drainage area relationships for the streams studied by detailed methods are shown in Table 2.

Table 2- Summary of Discharges

		Peak Discharges (cfs)			
Flooding Source and Location	**Drainage Area (square miles)**	**10-Percent-Annual-Chance**	**2-Percent-Annual-Chance**	**1-Percent-Annual-Chance**	**0.2-Percent-Annual-Chance**
BEAR CREEK					
At State Route 202	49.8	1,060	1,365	1,535	2,000
Above Evans Creek confluence	33.6	774	996	1,121	1,460
At River Mile 2.4	32.2	742	956	1,075	1,400
At N.E. 95th Street	30.1	710	915	1,028	1,340
At River Mile 3.5	29.3	689	887	998	1,300

Table 2 – Summary of Discharges *(Continued)*

Flooding Source and Location	Drainage Area (square miles)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
BEAR CREEK *(Continued)*					
Above Cottage Lake Creek confluence	14.7	320	460	520	690
Above Seidal Creek confluence	11.6	260	380	430	570
15 feet downstream of N.E. 145th Street	11.2	250	360	410	550
Above Struve Creek confluence	8.7	200	290	330	450
Above tributary confluence 3,200 feet upstream of N.E. 148th Street	8.0	190	270	310	410
1,500 feet downstream of Woodinville-Duvall Road	7.4	180	250	290	390
At Woodinville-Duvall Road	5.8	140	200	230	310
BIG SOOS CREEK					
At USGS gage 12-112600	66.7	1,130	1,440	1,550	1,790
Below Covington Creek confluence	49.4	870	1,110	1,190	1,380
Above Covington Creek confluence	31.2	580	740	800	920
Above Jenkins Creek confluence	13.5	270	350	390	450
Above Little Soos Creek confluence	9.3	200	250	280	320
At S.E. 244th Street	7.1	150	200	220	260
At S.E. 208th Street	4.5	100	130	150	170
BLACK RIVER					
Above Green River confluence	24.8	400[1]	400[1]	400[1]	400[1]
At P1 pump station inlet	24.8	650	1,040	1,230	1,730
CEDAR RIVER					
At USGS gage 12-119000	184	5,940	9,860	12,000	18,400
At 149th Avenue SE	--[2]	5,750	9,550	11,650	17,950
At Cedar Grove Road	--[2]	5,550	9,350	11,400	17,600
At Renton Maple Valley Road	--[2]	5,450	9,200	11,250	17,350
At State Route 18	--[2]	5,250	8,850	10,900	16,900
At Landsburg SE	121	4,880	8,340	10,300	16,100
COAL CREEK					
At mouth	7.31	228	306	340	420
At Interstate Highway 405	6.76	213	287	320	396

[1]400 cfs discharge from pump station coincides with peak flows in Green River
[2]Data Not Available

Table 2 – Summary of Discharges *(Continued)*

Flooding Source and Location	Drainage Area (square miles)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
COAL CREEK TRIBUTARY (NEWPORT CREEK)					
At mouth	0.31	14	21	25	35
COTTAGE LAKE CREEK					
At mouth	1	288	386	428	527
DES MOINES CREEK					
Below Marine View Drive South	5.8	400	600	702	945
EAST BRANCH OF WEST TRIBUTARY (KELSEY CREEK)					
At mouth	0.92	37	56	64	86
EAST FORK ISSAQUAH CREEK					
At mouth	9.5	560	900	1,050	1,980
EVANS CREEK					
Above Bear Creek confluence (including Bear Creek split-flow return)	--[1]	314	476	581	905
At River Mile 0.4	15.3	280	360	400	496
Near Redmond, at River Mile 0.8	13.0	280	360	400	496
FORBES CREEK					
At mouth	3.7	150	180	220	260
GARDINER CREEK					
At Northwest 8th Street	1.3	150	--[1]	300	--[1]
GILMAN BOULEVARD OVERFLOW					
At divergence from Issaquah Creek	--[1]	0.0	370	610	1,250
GREEN RIVER					
RM 44.3-40.4 Reach 1 (Upstream of Newaukum Creek)	--[1]	11,060	11,890	12,070	12,290
RM 40.4-33.3 Reach 2 (Newaukum Creek to Soos Creek)	--[1]	11,250	12,080	12,250	12,460
RM 33.3-23.7 Reach 3 (Soos Creek to Mill Creek	--[1]	11,230	12,420	12,810	13,460

[1]Data Not Available

Table 2 – Summary of Discharges *(Continued)*

Flooding Source and Location	Drainage Area (square miles)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
GREEN RIVER *(Continued)*					
Reach 4 (Mill Creek to Black River)			Varies by Subreach		
RM 23.7-21.5 Reach 4a (Downstream of Mill Creek Confluence	--[1]	11,580	12,500	12,530	12,610
RM 21.5-19.5 Reach 4b (Downstream of Mullen Slough	--[1]	11,700	12,650	12,690	12,800
RM 19.5-16.5 Reach 4c (Downstream of Midway Creek Confluence)	--[1]	11,740	12,700	12,750	12,870
RM 16.5-11.0 Reach 4d (Downstream of Tightline at RM 16.5)	--[1]	11,930	12,890	12,930	13,050
RM 11.0-0.0 Reach 5 (Black River to Mouth	--[1]	12,410	13,370	13,410	13,530
Above Howard A. Hanson Dam	215.0	20,050	29,250	33,500	49,000
HOLDER CREEK					
Above confluence with Carey Creek	7.5	420	660	800	1,150
ISSAQUAH CREEK					
At mouth	55.6	2,890	3,700	3,960	4,490
City Limit to Gage at 12121600	54.3	2,890	3,400	3,560	3,940
Through Gilman Bridge	49.4	2,570	3,320	3,550	4,000
Upstream of Gilman Overflow	49.3	2,570	3,690	4,160	5,250
Downstream of East Fork	49.2	2,560	3,670	4,140	5,230
Upstream of East Fork	39.7	2,080	2,980	3,360	4,230
KELSEY CREEK					
At mouth	10.10	301	398	439	536
At 140th Avenue N.E.	6.69	211	285	317	393
At Lake Hills Boulevard	2.25	84	121	138	179
LITTLE BEAR CREEK					
Above Sammamish River confluence	15.6	320	450	500	570
Above SR-202	15.5	340	490	570	750
At Highway 522	14.7	330	480	550	740
At N.E. 205th Street	13.6	310	450	520	700
LONGFELLOW CREEK					
At S.W. Brandon Street	2.7	170	310	380	520

[1]Data Not Available

Table 2 – Summary of Discharges *(Continued)*

Flooding Source and Location	Drainage Area (square miles)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
LONGFELLOW CREEK *(Continued)*					
At 26th Street S.W.	2.5	160	290	350	480
At S.W. Juneau Street	2.2	140	250	310	420
At 25th Avenue S.W.	2.1	130	240	290	400
At S.W. Willow Street	2.0	120	230	280	380
At S.W. Myrtle Street	1.4	84	150	180	250
At S.W. Webster Street (Detention basin outflow)	1.2	76	130	150	220
At S.W. Holden Street	1.1	74	120	140	200
LYON CREEK					
At mouth	3.67	147	177	188	214
MALONEY CREEK					
At Skykomish	3.8	750	980	1,130	1,380
MAY CREEK					
At USGS gage 12-119600	12.7	480	800	870	1,020
At Coal Creek Parkway	8.9	350	580	640	750
At 146th Avenue S.E.	7.7	310	520	560	660
At 148th Avenue S.E.	6.9	280	470	510	600
At 146th Avenue S.E.	4.8	200	340	370	440
At S.E. Renton-Issaquah Road	2.9	130	220	240	280
At S.E. May Valley Road	1.2	59	100	110	130
At S.E. 109th Place	0.9	46	78	87	100
MAY CREEK TRIBUTARY					
Above confluence with May Creek	1.5	72	120	140	160
McALEER CREEK					
At mouth	7.80	215	278	304	364
MERCER CREEK (INCLUDING BOTH MAIN AND RIGHT CHANNEL)					
At mouth	17.79	490	628	686	819
At confluence with Kelsey and Richards Creeks	13.75	393	510	560	675
MEYDENBAUER CREEK					
At mouth	1.33	133	150	160	177
At S.E. 6th Street	0.12	--[1]	--[1]	41	--[1]

[1]Data Not Available

Table 2 – Summary of Discharges *(Continued)*

Flooding Source and Location	**Drainage Area (square miles)**	**Peak Discharges (cfs)** 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
MIDDLE FORK LOWER OVERFLOW					
At divergence from Middle Fork	--[1]	200	1,600	2,300	4,200
Downstream of divergence of Middle Overflow	--[1]	100	1,100	1,400	2,600
MIDDLE FORK MIDDLE OVERFLOW	--[1]	100	500	900	1,600
MIDDLE FORK SNOQUALMIE RIVER					
At mouth	171.0	26,900	34,800	38,600	46,900
At Mt. Si Bridge	169.0	28,000	38,300	43,800	55,800
MIDDLE FORK UPPER NORTH OVERFLOW	--[1]	500	1,500	2,150	3,700
MIDDLE FORK UPPER SOUTH OVERFLOW					
At divergence from Middle Fork	--[1]	1,000	3,000	4,300	7,400
Downstream of divergence of Upper North Overflow	--[1]	500	1,500	2,150	3,700
MILL CREEK (AUBURN)					
Above confluence with Green River	12.8	250	360	410	510
At 277th Street	11.7	230/220	330/320	370/360	480/470
At 37th Street, N.W.	9.8	200/190	290/280	340/320	500/420
At 29th Street, N. W.	8.9	180	270	310	450
At Valley Freeway (SR-167)	8.0	180/170	270/250	310/280	500/400
At 15th Street, N.W.	7.6	190/170	300/250	370/290	570/480
At Main Street	6.2	160	250	310	490
At Peasley Canyon Way	5.7	140	230	290	450
At 15th Street, N.W.	0.7	--[1]	--[1]	40	--[1]
MILL CREEK (KENT)					
At confluence with Springbrook Creek	9.2	380	--[1]	650	--[1]
At Highway 167 culvert entrance	3.1	110	125	130	140
At Bowen-Scarff culvert outlet	2.9	110	115	120	130
Downstream of Springbrook Creek Overflow	2.7	85	90	100	110
At James Street	2.6	70	110	140	180

[1]Data Not Available

Table 2 – Summary of Discharges *(Continued)*

Flooding Source and Location	**Drainage Area (square miles)**	**Peak Discharges (cfs)** 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
MILLER CREEK *(Continued)*					
At mouth	8.1	383	575	670	1050
At sewage treatment plant	--[1]	278	415	479	785
At confluence with Lake Burien Tributary	--[1]	239	364	429	--[1]
Below 1st Avenue	--[1]	159	245	293	475
Below State Highway 509	--[1]	151	235	275	450
At confluence with Lake Lora Tributary	--[1]	109	176	211	--[1]
At Lake Reba outflow	--[1]	90	150	177	310
NORTH BRANCH MERCER CREEK (NORTH VALLEY CREEK)					
At mouth	3.10	111	157	177	227
At N.E. 40th Street	1.12	46	69	79	106
NORTH CREEK					
At mouth	30	958	1,290	1,440	1,810
Near Bothell (USGS gage No. 12-1260)	24.6	454	581	634	757
NORTH FORK ISSAQUAH CREEK					
At mouth	4.8	176	269	315	445
At mouth (including overtopping from Issaquah Creek)	4.8	176	489	835	1,995
NORTH FORK MEYDENBAUER CREEK					
At 102nd Avenue S.E.	1.03	94	105	113	128
NORTH FORK SNOQUALMIE RIVER					
At mouth	103.0	18,600	24,600	27,200	32,800
At North Bend gage	96.0	14,700	19,700	21,700	26,200
At Snoqualmie	64.0	12,300	16,300	18,000	21,700
NORTH FORK THORNTON CREEK					
Above South Fork Thornton Creek confluence	7.2	160	270	320	470
Below tributary confluence downstream of N.E. 115th Street	6.8	140	230	280	410

[1]Data Not Available

Table 2 – Summary of Discharges *(Continued)*

Flooding Source and Location	Drainage Area (square miles)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
NORTH FORK THORNTON CREEK *(Continued)*					
At N.E. 115th Street and 35th Avenue N.E.	5.6	90	150	180	270
At N.E. 125th Street	5.2	67	120	150	240
At 15th Avenue N.E.	4.2	42	82	110	170
At Interstate Highway 5	3.7	32	65	84	140
PATTERSON CREEK					
Snoqualmie River to Tributary 0377	--[1]	560	740	820	990
Tributary 0377 to Canyon Creek	--[1]	410	550	610	750
Canyon Creek to RM 4.56	--[1]	300	410	450	550
RM 4.56 to RM 5.92	--[1]	270	360	390	470
RM 5.92 to RM 7.77	--[1]	220	290	320	380
RM 7.77 to Redmond-Fall City Road	--[1]	160	220	240	300
Upstream of Redmond-Fall City Road	--[1]	90	130	150	180
RAGING RIVER					
At mouth	32.9	4,031	6,286	7,413	10,465
At USGS gage 12-145500	30.6	3,790	5,910	6,970	9,840
Above Interstate Highway 90	25.7	3,268	5,095	6,009	8,483
Above Lake Creek confluence	20.2	2,652	4,135	4,877	6,885
Above Deep Creek confluence	13.3	1,851	2,887	3,404	4,806
RICHARDS CREEK					
At mouth	3.63	122	170	191	241
At Interstate Highway 90	1.11	44	65	75	99
At S.E. Newport Way	0.80	33	50	58	78
RICHARDS CREEK EAST TRIBUTARY					
Approximately 325 feet upstream of S.E. 26th Street[2]	0.06	4	36	47	81
RICHARDS CREEK WEST TRIBUTARY					
At mouth	0.91	37	55	64	85

[1]Data Not Available
[2]Includes overflow from Richards Creek for 2-, 1-, and 0.2-percent-annual-chance discharge

Table 2 – Summary of Discharges *(Continued)*

Flooding Source and Location	Drainage Area (square miles)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
ROLLING HILLS CREEK					
At Highway 405 culvert entrance near Highway 167	1.2	72[1]	86[1]	91[1]	--[2]
Below east storm drain confluence 600 feet upstream of Highway 405	1.2	77	110	130	--[2]
SAMMAMISH RIVER					
At confluence with Lake Washington	--[2]	3,980	4,930	5,300	6,130
Just downstream of the confluence of Bear Creek	--[2]	1,980	2,420	2,590	2,970
SNOQUALMIE RIVER					
At Duvall	--[2]	53,400	75,800	84,600	99,700
At Carnation	603.0	58,200	82,400	91,800	113,300
Near Snoqualmie	375.0	51,700	71,000	79,100	95,200
SOUTH FORK SKYKOMISH RIVER					
At Index gage	355.0	44,300	65,200	74,700	98,500
At Baring	336.0	42,300	62,200	71,300	94,000
Just upstream of Miller Creek	245.0	32,200	47,400	54,300	71,600
Just upstream of Beckler River	139.0	12,600	19,400	22,800	31,700
SOUTH FORK SNOQUALMIE RIVER					
At mouth	86.8	10,100	16,500	20,200	28,600
At North Bend gage	81.7	9,000	13,000	15,000	19,700
At Edgewick gage	65.9	8,900	12,900	14,900	19,500
SOUTH FORK THORNTON CREEK					
At 35th Avenue N.E. and N.E. 105th Street	3.8	150	230	270	380
At 30th Avenue N.E.	3.6	140	210	250	350
At Lake City Way	3.2	120	180	210	300
At N.E. 107th Street	2.1	72	110	130	180
At N.E. 105th Street and 8th Avenue N.E.	1.4	50	75	89	120
SPRINGBROOK CREEK					
Upstream of confluence with Black River	21.9	590[3]	930	1,100[3]	1,550

[1]Downstream decrease in discharge results from routing effects of hydraulic structures
[2]Data Not Available
[3]Decrease in discharges due to P1 pumping plant pumping 300 cfs into Green River flood stages

Table 2 – Summary of Discharges *(Continued)*

Flooding Source and Location	Drainage Area (square miles)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
SPRINGBROOK CREEK *(Continued)*					
Downstream of confluence with Mill Creek (River Mile 3.03)	16.1	680[1]	--[2]	1,055	--[2]
SWAMP CREEK					
At USGS gage 12-127100	23.1	600	810	910	1,160
At tributary confluence downstream of 73rd Avenue N.E.	21.9	570	770	870	1,110
At N.E. 205th Street	20.9	550	740	830	1,060
THORNTON CREEK					
Above mouth at Lake Washington	12.1	190	290	390	670
At N.E. 93rd Avenue	11.7	150	230	330	590
At 45th Avenue N.E.	11.5	140	210	310	560
At N.E. 105th Street	11.1	110	170	260	490
At diversion weir to downstream channel	11.0	100	160	250	480
At diversion to Lake Washington	--[2]	210	330	340	350
Below confluence of North and South Fork Thornton Creek	10.9	310	490	590	830
TIBBETTS CREEK					
At mouth	3.9	220	355	425	600
At confluence with West Fork Tibbetts Creek	2.8	183	254	286	367
At approximately 600 feet upstream of Southwest 83rd Place	1.9	129	180	203	261
TOLT RIVER					
At mouth	97.0	13,900	19,500	22,000	27,800
At USGS Gage 12148500 (near Carnation)	81.4	11,900	16,700	18,800	23,800
UNNAMED DRAINAGEWAY					
In the central business district in the city of Kirkland	1.5	--[2]	--[2]	--[2]	--[2]
VASA CREEK					
At mouth	1.37	55	81	93	123
At cross section R	0.53	24	38	44	60

[1]400 cfs discharge from pump station coincides with peak flows in Green River

[2]Data Not Available

Table 2 – Summary of Discharges *(Continued)*

Flooding Source and Location	**Drainage Area (square miles)**	**Peak Discharges (cfs)** 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
WALKER CREEK					
Above confluence with Miller Creek	1.5	281	400	461	605
WEST FORK ISSAQUAH CREEK					
Above Issaquah Creek confluence	4.9	290	460	550	790
2,900 feet upstream of 229th Drive S.E.	4.7	270	440	530	770
Above tributary confluence near 208th Avenue S.E.	1.5	100	160	200	280
WEST TRIBUTARY KELSEY CREEK					
At mouth	1.75	64	92	104	135
At upstream confluence of East Branch	0.34	16	25	29	41
WHITE RIVER					
At Pacific and Auburn	454	14,000	15,300	15,500	19,000
YARROW CREEK					
At mouth	2.2	--[1]	--[1]	126	--[1]
At unnamed drainageway in Central Business District	1.5	--[1]	--[1]	339	--[1]
At N.E. 40th Street	0.73	29	44	41	68

[1]Data Not Available

The USACE regulates the water level of Lake Washington at the Hiram M. Chittenden Locks on the Lake Washington Ship Canal. The lake level is drawn down during the winter months and is typically regulated at elevation 16.8 NAVD for that period.

In the summer months, the lake level is raised to an elevation of 18.6 feet NAVD. That elevation exceeds the normal depth water-surface elevation determined at the mouth of the Cedar River for the 10-, 2-, and 1-percent-annual-chance recurrence interval flows. Therefore, the flood profiles for the Cedar River includes the backwater impact from Lake Washington until the profile that was started at normal depth exceeds the 15.0-foot elevation for the 1-percent-annual-chance recurrence interval event at the first cross section, with lake backwater shown for the lesser recurrence intervals.

Elevations on Lake Sammamish for the various frequency floods are controlled by the USACE Lake Sammamish outlet project built in 1966.

This project consists of a low weir designed to maintain the lake elevation at 32.6 feet for the 10-percent-annual-chance flood. The elevations for the 2-, 1-, and 0.2-percent-annual-chance floods were computed by routing techniques through the lake. Elevations for floods for the selected recurrence intervals are also presented in Table 3.

Table 3 – Summary of Elevations
Elevation in Feet (NAVD 88)

Flooding Source and Location	10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
Lake Sammamish	34.5	35.8	36.2	37.3
Phantom Lake	*	*	263.8	*

*Data Not Available

3.2 Hydraulic Analyses

Analyses of the hydraulic characteristics of flooding from the sources studied were carried out to provide estimates of the elevations of floods of the selected recurrence intervals. Users should be aware that flood elevations shown on the FIRM represent rounded whole-foot elevations and may not exactly reflect the elevations shown on the Flood Profiles or in the Floodway Data tables in the FIS report. Flood elevations shown on the FIRM are primarily intended for flood insurance rating purposes. For construction and/or floodplain management purposes, users are cautioned to use the flood elevation data presented in this FIS in conjunction with the data shown on the FIRM.

Cross section data for the backwater analysis for Miller Creek, Walker Creek, and a portion of Des Moines Creek were taken from topographic maps with 2 foot contour intervals (Reference 44). Cross section data for North Creek and White River (left bank overflow) were taken from aerial photographs (References 45 and 46). Cross section data for the Snoqualmie River and North, Middle, and South Forks Snoqualmie River were obtained from field surveys and aerial photographs (References 1, 47, 48, and 49). The cross section data for the backwater analyses for the remaining streams studied by detailed methods were obtained by field survey. Cross section data for the overbank areas of Green River, Tibbetts Creek, Issaquah Creek, and East Fork Issaquah Creek were based on topographic base maps (References 50 and 51).

The flooding potential, in the form of ponding, for the unnamed drainageway in the central business district in Kirkland, is directly related to the capacity of the existing stormwater drainage system. The capacity of this system was determined and removed from the runoff produced by the design storm. The volume of the remaining excess runoff was then compared to a storage-elevation curve developed for the central business district. This comparison yielded the maximum expected elevation for the predicted 1-percent-annual-chance event. Based on a Letter of Map Revision (LOMR) dated January 30, 1989, and due to improvements done in that area, the drainageway was moved to reflect the LOMR.

Water-surface elevations of floods of the selected recurrence intervals on Mercer Creek, Right Channel Mercer Creek, Meydenbauer Creek, North Fork Meydenbauer Creek, Coal Creek, Vasa Creek, Richards Creek East Tributary, Richards Creek West Tributary, Kelsey Creek, West Tributary Kelsey Creek, East Branch of West Tributary Kelsey Creek, North Branch Mercer Creek, and Yarrow Creek were computed using the USGS E-431 step-backwater computer model (Reference 52). Water-surface elevations of floods of the selected recurrence intervals on Lyon Creek and McAleer

Creek downstream of Northeast 178th Street were computed by hand calculations.

McAleer Creek passes through 10 significant hydraulic structures, one private culvert, and numerous private bridges. Lyon Creek passes through 12 significant hydraulic structures. Each of these structures was rated for hydraulic capacity by applying standard hydraulic calculations and hydraulic nomographs (References 53, 54, 55, and 56).

The water-surface elevations for a portion of the upper Green River Valley were computed using the USACE G3722110 Water-Surface Profiles computer program (Reference 57).

The water-surface elevations of floods of the selected recurrence intervals on the remaining streams studied by detailed methods were computed using the USACE HEC-2 step-backwater computer program (Reference 58).

The starting water-surface elevations for the Snoqualmie River and North, South, and Middle Forks Snoqualmie River, Sammamish River, Tibbetts Creek, and Green River were developed using the slope-area method or were developed from hydraulic rating data. For the most downstream portion of the Green River, the starting water-surface elevation was based on previous studies. The starting water-surface elevation of 6.6 feet, which lies below the highest estimated tide and above the mean high water elevation, was calculated by the USACE with the coordination of FEMA Region X.

Starting water-surface elevations for Raging, Cedar, and South Fork Skykomish River, and Big Soos, Swamp, Issaquah, West Fork Issaquah, Thornton, Longfellow, Forbes, Yarrow and Maloney Creek were determined using normal depth from slope-area methods.

Starting water-surface elevations for Rolling Hills Creek and May Creek were determined to be critical depth. Starting water-surface elevations for May Creek Tributary were the corresponding recurrence interval event water-surface elevations in the main stem at the point of confluence with the tributary.

The starting water-surface elevations for Bear and Evans Creeks are coincident with the elevations at the confluences of the Sammamish River and Bear Creek, respectively.

Starting water-surface elevations for the White River were taken from the USACE computer printout and flood profiles prepared in 1974 (Reference 40).

The starting water-surface elevation for Lyon Creek and McAleer Creek was the maximum control elevation of Lake Washington, which is 15 feet.

The starting water-surface elevation for North Creek at its mouth was the 10-percent-annual-chance flood elevation from the Sammamish River.

Starting water-surface elevations for Little Bear Creek were based on a coincident 25-year recurrence interval Sammamish River flood stage, as was estimated to occur for the January 1986 flooding event. The starting water-surface elevation for Mill Creek (Auburn) was based on computed Green River backwater elevations at the Mill Creek outlet using mean monthly Green River flow data for December and January.

The starting water-surface elevation on Mill Creek (Kent) was obtained from the Springbrook Creek flood profile.

Starting water-surface elevations for the flood profiles for Miller Creek and Walker Creek were taken from the hydraulic study of Puget Sound. Starting elevations for the flood profiles for Des Moines Creek were taken using the 10-percent-annual-chance elevation computed for Puget Sound.

For the coastal area studied by detailed methods, the effects of high tidal levels and wave runup were combined to determine the maximum flood elevations above the NAVD 1988 datum. Wave prediction and wave runup calculations were performed by methods prescribed in the USACE Shore Protection Manual (Reference 59).

Starting water-surface elevations for Mercer, Right Channel Mercer, Meydenbauer, North Fork Meydenbauer, Coal, Vasa, Richards, East Tributary Richards, West Tributary Richards, Kelsey, West Tributary Kelsey, East Branch of West Tributary Kelsey, and North Branch Mercer

Creeks were computed form:

1. Frequency analysis of lake elevations
2. Profile conveyance of downstream cross sections
3. Culvert ratings where an approach section was the section farthest downstream

The starting water-surface elevations for the Black River, North and East Forks Issaquah Creek, and North and South Forks Thornton Creek are coincident with the elevations at the confluences of the Green River, Issaquah Creek, and Thornton Creek, respectively.

For the Green River, analyses were performed in accordance with FEMA's levee policy. In accordance with those guidelines, two backwater profiles were computed for the reach under study, one for flows confined to the levee system, and a second for the condition of complete levee systems assumed removed for analysis, where levee system freeboard is less than minimum FEMA standards. The general freeboard standard of 3 feet for consideration of levee flood protection was lowered by FEMA for the Green River to 2 feet based on USACE review and recommendations, at the request of King County (Reference 60). Based on the computed with levees water-surface profiles and surveyed cross section and levee profile data, a total of approximately 5.7 river miles of levees were identified as having less than 2 feet of freeboard at some locations along a particular levee system.

On Little Bear Creek, high water marks for the January 1986 event were used to calculate flows through culverts and to reduce flows at overbank breakout points, from upstream of the SR 202 culvert, downstream to the Sammamish River confluence. The HEC-2 step-backwater model was calibrated to these conditions. A range of flows were input to the model to develop rating curves for the structures and overflow weirs. The recurrence flows, derived from the hydrologic analyses, were modified to reflect the overflow conditions from review of the rating curves. Sheetflow and ponding caused by the channel overflow was approximated from photographs, topographic maps, high water marks, and local accounts of flooding extent and depths.

The maximum water-surface elevation of the P1 storage pond in Renton was determined by routing the hydrograph through the storage pond and pumping station by using the storage-elevation relationship for the pond and the pumping station's firm capacity of 875 cfs as the maximum discharge. The 10-percent-annual-chance water-surface profile for Springbrook Creek was started at normal depth because normal depth was greater than 3.5 feet NGVD, which is the maximum water-surface elevation of the P1 storage pond under standard operating procedures. The peak 10-percent-annual-chance flow into the storage pond is less than the maximum pumping rate and, therefore, no rise in the water-surface elevation of the storage pond should occur during the 10-percent-annual-chance event. Two conditions were considered for each of the 2-, and 1-percent-annual-chance events. The first consisted of modeling the effects on the Springbrook Creek study reach of the computed maximum water-surface elevation that may be reached in the storage pond (the starting water-surface elevation) coincident with the flow that would be discharged from Springbrook Creek at that time step in the inflow runoff hydrograph. The second condition of analysis consisted of modeling the effects of Springbrook Creek peak inflows for the recurrence interval event under consideration, with a starting water-surface elevation of the higher of normal depth, or the coincident elevation of the storage pond at the time of the peak inflow. For each recurrence interval, the higher water-surface elevation resulting from each of those analysis conditions at the study reach cross sections was used for final flood profile determination.

Areas of coastline subject to wave attack are referred to as coastal high hazard zones. Factors considered in determining wave runup included length of fetch, sustained wind velocities, coastal water depths, land slopes, and other physical features of the coastline that could appreciably affect wave propagation. Much of the coastline along Des Moines is protected by a breakwater that extends north and south along the coast to protect the Des Moines Marina. The area west of this breakwater and the unprotected area north and south of the breakwater have been designated coastal high hazard zones. The unprotected sections of the coastline are subject to wave attack generated by high winds from a southwest direction across Puget Sound. The remaining coastal areas inland from the breaking waves, subject only to wave runup, and areas sheltered by the breakwater are not exposed to severe wave attack and have not been designated as part of a coastal high hazard zone.

Channel and overbank roughness factors (Manning's "n") used in the hydraulic computations were chosen by engineering judgment and were based on field observations of the stream and floodplain areas, and hydraulic calibration of flood profiles to available high water mark data. The range of channel and overbank "n" values for the various flooding sources are listed in Table 4.

Flood profiles were computed an accuracy of approximately 1 foot for floods of the selected recurrence intervals and are shown in Exhibit 1. The degree of accuracy of the water-surface profiles is limited to 1 foot by the location and accuracy of the cross sections, the extent of the various energy losses of the system, and the general limitations of backwater calculations. The accuracy of 1 foot is consistent with the accuracy of predicted peak discharges and the knowledge that unpredictable events during actual floods will likely cause deviations from the predicted profile.

Locations of selected cross sections used in the hydraulic analyses are shown on the Flood Profiles (Exhibit 1). For stream segments for which a floodway was computed (see Section 4.2), selected cross section locations are also shown on the Flood Insurance Rate Maps (Exhibit 2).

For streams studied by approximate methods, the 1-percent-annual-chance floodplains were approximated by field inspections and observations and by normal depth calculation using estimated 1-percent-annual-chance recurrence interval floodflows and approximate cross sections taken from field investigations or from topographic maps, where available. Computed depth from minimum channel elevation and average floodflow velocity are shown on the maps.

The hydraulic analyses for this study were based on unobstructed flow. The flood elevations shown on the profiles are thus considered valid only if hydraulic structures remain unobstructed, operated properly, and do not fail.

3.2.1 Revision 1

Miller Creek - Analyses of the hydraulic characteristics of Miller Creek were carried out to provide estimates of flood elevations for the 10-, 2-, 1-, and 0.2-percent-annual-chance events. Water-surface elevations were computed using the September 1990 release of the USACE HEC-2 backwater computer program (Reference 97). Data required to develop the HEC-2 model include channel and floodplain geometry, roughness coefficients, and starting water-surface elevations. Cross-section data for the backwater analyses were obtained from field surveys performed between November 1990 and January 1991. A total of 32 sections were surveyed. All significant bridges, culverts, and weirs were surveyed to obtain elevation data and structural geometry. A total of six bridges, eight culverts, and 11 weirs were surveyed.

In the HEC-2 program, the special bridge routine was used for bridges with piers and for those where pressure flow occurred. The normal bridge routine was used for bridges without piers and for low-flow conditions where the water surface was below the low-chord elevation of the bridge. Local residents have built a number of small, wooden foot bridges across the creek. These were not included in the model.

Water-surface elevations at each culvert were also computed using the HEC-2 model, which incorporated the capability to simulate culvert hydraulics using Federal Highway Administration culvert procedures. For weir flow, water-surface elevations at each weir were computed using the HEC-2 model. The geometry of each weir was defined in the model, and water-surface elevations were computed using standard step-backwater analyses.

Channel roughness (Manning's "n") values used in hydraulic computations were determined using engineering judgment, reference to classical publications (References 98 and 99), and calibration to observed conditions. Flood profiles were matched with high-water marks and discharge data collected during January and February 1991 events. Selected channel "n" values range from 0.040 to 0.057, and overbank values range from 0.070 to 0.110.

The starting water-surface elevation was calculated using the slope-area method, based upon an assumed water-surface slope of 0.003.

Tub Lake Tributary flows from a depression area south of Beverly Park along Des Moines Way heading south. It then empties into the Lake Reba Detention Pond through a culvert underneath State Highway 518. Because this is a minor tributary to the mainstem of Miller Creek, approximate methods were used to assess the flood hydraulics. This tributary consists of approximately 1,300 feet of open channel and 250 feet of piped segments. From its confluence with Miller Creek, the tributary begins as an open channel. Approximately 900 feet upstream, a 200-foot long, 18-inch-diameter steel pipe carries flow under a little league baseball field. Upstream, 400 feet of open channel carry flow from a 240-inch-diameter corrugated metal pipe (CMP) culvert that conveys flow under South 144th Street. The Tub Lake marsh area begins north of South 144th Street. Both open channel reaches are represented in the HEC-2 model by a trapezoidal cross section that has a 4-foot depth, a 4-foot bottom width, and 2H:1V side slopes. Channel and floodplain geometry used in the model were estimated from available topographic mapping and data collected during a site reconnaissance.

Channel roughness coefficients were assumed to be 0.065 for open channel, 0.070 for overbanks, 0.015 for the steel culvert, and 0.024 for the CMP culvert.

The hydraulic analyses for this study were based on unobstructed flow. The flood elevations shown on the profiles are thus considered valid only if hydraulic structures remain unobstructed, operate properly, and do not fail.

Elevation Reference Marks (ERMs) were established at eight sites along the stream. Floodplain boundaries were delineated in the detailed study reach of Miller Creek and its tributary using topographic maps at a scale of 1:2,400, with a 5-foot contour intervals, provided by the King County Department of Public Works and the City of Seattle Engineering Department.

The floodways developed in this study were computed with the HEC-2 model, generally with the assumption of equal-conveyance reduction from each side of the floodplain. Floodway widths were computed at each cross section. Between sections, the floodway boundaries were interpolated. The results of the floodway study are tabulated for each cross section in Table 6, "Floodway Data." No floodway was computed for the Tub Lake Tributary.

The information for this restudy of Miller Creek supersedes the data presented in the previous Flood Insurance Study for King County, dated September 29, 1989 (Reference 94). The discharges used in this study of Miller Creek were revised to account for the effects of urbanization and operations of the newly constructed Lake Reba Detention Pond. This restudy was completed in September 1991.

3.2.2 Revision 2

Snoqualmie River - Nhc compared the two hydraulic studies performed by Hosey & Associates for Puget Power and measured high-water marks with the profiles published by FEMA for the Snoqualmie River in the vicinity of the City of Snoqualmie. The more recent of these two studies incorporated updated topographic information and was calibrated using information from recent storms. When the profiles produced by these studies matched FEMA's profile, it was determined that a restudy of the area was not warranted at that time. However, upon comparison between the base (1-percent-annual-chance) flood elevation (BFE) placements shown on Flood Insurance Rate Map Panels 53033C0737 K, 53033C0739 K, 53033C0741 K, and 53033C0743 K and those shown on the published profile, it was determined that the BFE placements shown on the above-mentioned Flood Insurance Rate Maps were incorrect. Therefore, the BFE placements shown on the above-mentioned Flood Insurance Rate Map panels were revised along the Snoqualmie River from approximately 1,530 feet upstream of State Highway 202 to its confluence with the South Fork Snoqualmie River to match those shown on the published profiles for that reach.

3.2.3 Revision 3

Raging River - The hydraulic analysis for the revised study of the downstream reach was performed using the USACE HEC-2 backwater computer program (Reference 97). Data for the cross sections, including overbank areas, were taken from field surveys performed in April 1993. A total of 52 sections were surveyed, including seven bridges. There are additional bridges along the Raging River that were not modeled because they do not affect the water-surface elevations of the river.

Channel and overbank roughness coefficients (Manning's "n") used in the computer program for the downstream reach were estimated from experience and field observations. Values range from 0.035 to 0.055 in the channel and from 0.050 to 0.090 in the overbank areas.

The starting water-surface elevation was obtained by the slope-area method based on an estimated slope of the energy-grade line.

Downstream of 328th Way to the confluence with the Snoqualmie River, the Raging River is confined between levees. However, these levees do not meet FEMA freeboard requirements. Therefore, the water-surface profiles for the area affected by the levees were computed as follows:

1. For the area between the levees, the profiles were determined considering that both levees would remain in place.

2. For the right overbank (looking downstream), the profiles and floodplain boundary were determined without considering the effects of the right levee.

3. For the left overbank, the profiles and floodplain boundary were determined without considering the effects of the left levee.

For the upstream reach, the revised discharge values were used to complete a revised hydraulic analysis using HEC-2 and the cross-section information and Manning's "n" values from the previous

Flood Insurance Study. The water-surface elevations increased by a maximum of 4.7 feet approximately 0.6 mile upstream of I-90 and the floodplain width increased by a maximum of 120 feet approximately 1.3 miles upstream of I-90.

The 1-, and 0.2-percent-annual-chance floodplain boundaries for both the upstream and downstream reaches were delineated using the flood elevations determined at each cross section. Between cross sections, the boundaries were interpolated using topographic maps at a scale of 1:2,400, with a contour interval of 2 feet (Reference 100) for the downstream reach. The topographic work maps (Reference 65) from the previous Flood Insurance Study were used to delineate the floodplain boundaries between cross sections for the upstream reach. In cases where the lines are collinear, only the 1-percent-annual-chance flood boundary has been shown.

The floodway determined for the Raging River was computed based on equal conveyance reduction from each side of the floodplain, and in the floodplain area downstream of 328th Way, the floodway was determined without consideration of the levees. Floodway widths were computed at each cross section. Between sections, the floodway boundaries were interpolated. In cases where the floodway line is collinear with the 1-percent-annual-chance floodplain line, only the floodway line has been shown.

Locations of selected cross section used in the hydraulic analyses are shown on the Flood Profiles (Exhibit 1).

The hydraulic analyses for this study were based on unobstructed flow. The flood elevations shown on the profiles are thus considered valid only if hydraulic structures remain unobstructed, operate properly, and do not fail.

3.2.4 Revision 4

North Fork Issaquah Creek - Updated estimates of Issaquah Creek 1-percent-annual-chance elevations affecting the North Fork channel have been reported by the City of Issaquah in 1992, based on a HEC-2 model that was calibrated to high-water marks for the January 1990 flood (Reference 103).

Estimates of Issaquah Creek overbank flow entering the North Fork channel were made by assuming weir flow in two segments that correspond to relatively low sections along the channel banks.

The first (upstream) section was represented as a 500-foot-long weir located between Cross Sections C and D. The second (downstream) section was represented as a 200-foot-long weir between Cross Sections B and C. Average depths of flow over these sections under 1-percent-annual-chance flood conditions were estimated to be 0.5 and 0.3, respectively. Depths of flow for 2-, and 0.2-percent-annual-chance events were estimated to be approximately 0.2 foot lower and 0.5 foot higher, respectively, than the 1-percent-annual-chance flow depths. A broad-crested weir coefficient of 2.5 was assumed for computing overbank flow. Approximately 440 cfs additional flow enters North Fork Issaquah Creek from Issaquah Creek between Cross Sections C and D, and approximately 80 cfs enters between Cross Sections B and C. The floodway analyses considered only the basin flows and did not include additional flows due to overtopping.

A detailed backwater model was created for the entire study reach using the February 1991 release of HEC-2 (Reference 104). An existing HEC-2 model of the lower portion of the study reach was obtained from King County and modified for purposes of the restudy.

The physical geometry of the North Fork Issaquah Creek channel was represented by 11 cross sections surveyed in 1989 and 1994. Channel cross sections were surveyed in April and May 1989 by David Evans and Associates (DEA) for King County at six locations from the mouth to just upstream of SE 64th Place. An additional five channel cross sections were surveyed in October and November 1994 by nhc to define the upstream portion of the study reach.

Floodplain geometry was estimated from a 2-foot contour mapping obtained from the City of Issaquah Department of Public Works in digital and hard-copy format. The contour mapping was prepared by David C. Smith and Associates of Portland, Oregon, based on photography dated April 11, 1989.

Eight bridges, one rectangular weir, and a complex multiple-culvert crossing at the I-90 interchange are represented in the North Fork Issaquah Creek HEC-2 model. The data to define these structures were obtained from DEA surveys made for the lower portion of the study reach in 1989, from nhc field surveys made for the upper portion of the study reach in 1994, and from construction drawings for the I-90 interchange obtained from the WSDOT.

A small footbridge located approximately 20 feet upstream of the rectangular weir in the upstream portion of the study reach was not represented in the model. The footbridge spans the full channel without any fill or encroachments, and appeared unlikely to survive a major flood.

Approximate methods were sued to assess the complex culvert crossing at the I-90 interchange. The existing crossing consists of an original dual-culvert system that was augmented by a large bypass culvert after the original system failed to perform satisfactorily.

The original I-90 crossing design was constructed in 1968/1969. It is a complex design with three sections of dual 42-inch- and 54-inch-diameter culverts at different invert elevations and slopes, alternating with two open-water sections in the areas enclosed by on and off ramps between I-90 and East Lake Sammamish Parkway. In each of the dual-culvert segments, one of the two culverts is constructed with zero slope. Sediment obstruction of the upstream (3.5-foot-diameter) zero-slope culvert is believed to have been a major cause of upstream flooding following completion of the original crossing design. The I-90 crossing design was substantially modified in 1973, with the addition of a single 260-foot-long, 66-inch-diameter bypass culvert beneath East Lake Sammamish Parkway.

The complex crossing at the I-90 interchange is represented in the HEC-2 model by an equivalent culvert that was determined using the WSDOT's HY-8 culvert program. In determining an equivalent culvert, it was assumed that the zero-slope culvert from the original design is completely ineffective due to sediment obstruction, consistent with verbal reports that such blockage has occurred during past flood events. All remaining culverts were assumed to be in good hydraulic condition and free of blockages.

Individual rating curves based on a constant (approximately 1-percent-annual-chance) tailwater level of 66.5 feet were determined for the two active flow paths, and manually summed to derive a composite rating curve. An equivalent culvert was then determined by trial and error so that the equivalent rating curve matches the composite rating curve at the 1-percent-annual-chance discharge.

The equivalent culvert used in the HEC-2 model is a single 6.3-foot-diameter culvert that is 250 feet long and follows the alignment and slope of the bypass culvert under East Lake Sammamish Parkway.

Channel roughness values (Manning's "n") in the HEC-2 model were determined by calibration to observed water levels and by reference to USGS Water Supply Papers 1849 and 2339, which discuss roughness characteristics of natural channels and floodplains (References 105 and 106).

Manning's "n" values ranged from 0.03 to 0.12 for channel sections and from 0.06 to 0.20 for overbank areas. The highest channel roughness values correspond to reaches of the channel having well-established trees and other vegetation within the sections coded in the HEC-2 model as being the main channel section. The values presented in the model are reasonable in relation to values presented by the USGS (1978 and 1989) (Reference 106).

Inundated areas that do not convey flow were assigned "n" values of 0.99 or higher. High "n" values were defined during the hydraulic analysis of the 1-percent-annual-chance flood condition and were used to balance the horizontal distribution of main-channel and overbank flows, with consideration of contraction and expansion of flow upstream and downstream of bridge crossings.

Starting water-surface elevations for the analysis assume coincident peak water levels in the main stem Issaquah Creek channel. Coincident peaks were assumed because 1-percent-annual-chance flood conditions in the lower reach of North Fork Issaquah Creek will be dominated by flows that originate from the main stem channel.

There are floodplain boundary discontinuities between the North Fork Issaquah Creek and main stem channels in the vicinity of Cross Section D. Issaquah Creek floodplain boundaries through this reach were last studied in 1977. Most of the Zone X areas between the North Fork Issaquah Creek and main stem channels are subject to inundation during a 1-percent-annual-chance flood on Issaquah Creek.

The normal depth of flow was used to determine the starting water-surface elevation for the floodway analysis.

The flood risk in the upper study reach from SE 66th Street to the downstream crossing at the I-90 interchange is highly dependent on culvert maintenance at the I-90 interchange and on channel aggradation upstream from the rectangular weir located in this reach. At the I-90 interchange, it was assumed for the restudy that the zero-slope culvert of the original design is fully obstructed, but that the second (sloping) culvert from the original design plus the bypass culvert are both maintained to be in good hydraulic condition. It was further assumed that the channel from the SE 66th Street bridge to the rectangular weir is not prone to aggradation, which would cause a significant reduction of the channel capacity.

The floodway boundaries developed in the restudy were computed with the HEC-2 model based on the North Fork basin 1-percent-annual-chance discharge of 315 cfs, which excludes any additional flow originating from the main stem Issaquah Creek. The starting water level for the encroachment analyses was set at 1 foot above the normal depth of flow for the 1-percent-annual-chance discharge of 315 cfs.

The stream has a small active channel, typically approximately 10 feet wide and 3 feet deep, which is contained within a larger main channel that is typically approximately 35 to 50 feet wide and 8 feet deep. Top-of-bank stations in the HEC-2 model are coded to reflect the smaller active channel in order to recognize substantial variations in roughness across the larger main channel. However, top-of-bank stations corresponding to the larger main channel are more appropriate in the context of determining minimum floodway widths.

The floodway width and other floodway data that correspond to encroachment limits set at the top of the main channel banks were incorporated in Table 6, “Floodway Data.”

The profile for North Fork Issaquah Creek was revised as a result of the restudy.

Bear Creek - The restudy of Bear Creek included detailed and approximate hydraulic analyses to estimate floodplain and floodway boundaries along the entire study reach. Detailed methods were used to determine the floodway boundaries and estimate the majority of the floodplain along Bear Creek. Approximate methods were used to determine depths of flow and inundation limits in the overbank area associated with a flow split downstream of NE 95th Street near the Friendly Village mobile-home park. At the upstream end of the study reach, the detailed hydraulic analyses were extended approximately 420 feet upstream of Avondale Road NE to tie into the previous study (Reference 94).

Because discharges for intermediate points along the main stem of Bear Creek appeared unreasonable in the previous study, new discharges were computed based on a combination of the peak flows at the mouth of Bear Creek and the distribution of flows across the study reach computed by Entranco Engineers, Inc., in the 1993 HSPF hydrologic analysis (Reference 108). To determine the flow at any point along Bear Creek, the appropriate recurrence interval flow at the downstream end of the study reach, from the previous study, was multiplied by the ratio of discharges at the two locations from the Entranco analysis. Discharges along Bear and Evans Creeks were incorporated in Table 2, "Summary of Discharges."

Evans Creek - Discharges from the previous study dated September 1989 were used directly at three locations in the HEC-2 model: at the downstream end of the study reach on Bear Creek, near the mouth of Evans Creek, and at the upstream end of the study reach above the confluence with Cottage Lake Creek. Discharges at other points in the study reach were recomputed after review of the previous model indicated discharges at intermediate points were not consistent or reasonable.

Cottage Creek - The hydraulic analyses performed for the restudy only extended up Cottage Lake Creek approximately 150 feet to include the entire width of the floodplain shared jointly by the two creeks. No further analysis or floodplain mapping was performed for Cottage Lake Creek

The KCSWM developed backwater models of Bear, Evans, and Cottage Lake Creeks for the 1990 Bear Creek Basin Plan using the USACE HEC-2 model (Reference 104).

The HEC-2 model was modified as follows:

- More detailed cross-section data from a recent LOMR issued April 28, 1994, on lower Bear Creek were substituted for the King County data for the reach between State Route 202 and Union Hill Road (Reference 109).

- The representation of the Union Hill Road bridge was updated to reflect the construction of a new bridge in 1994.

- The KCSWM HEC-2 model was augmented with additional detailed cross-section data from a 1986 hydraulic investigation for the reach between Union Hill Road and the Redmond Animal Clinic (Reference 110).

- Encroachment cards in the KCSWM HEC-2 model, used to limit effective flow areas at bridges, were replaced with NH cards to facilitate floodway analyses. The locations of the fully expanded flow sections were also adjusted consistent with recommendations in the HEC-2 User's Manual.

- The model configuration at several bridges was updated to more accurately simulate roadway overtopping and corresponding hydraulic losses.

- Split-flow analyses were included to represent areas in the Bear and Evans Creeks models where significant flow exits the main channel and flows in a hydraulically separate flow path before returning to the main channel downstream.

- The Bear Creek model was updated to reflect recent bridge replacements on Avondale Road NE at the two most upstream crossings, approximately at River Miles 5.4 and 5.7. Updated bridge geometry was based on nhc field surveys performed in August 1995.

- The model was calibrated to high-water marks from the January 18, 1986, and January 10, 1990, flood events. Calibration led to modifications in Manning's "n" roughness values and the addition of several intermediate cross sections.

The physical geometry of the Bear Creek channel was represented by 74 surveyed cross sections. These cross sections were developed primarily by the KCSWM based on field surveys by DEA in 1987. Surveyed cross sections were extended by the KCSWM using the 1987 aerial topographic mapping prepared by David Smith and Associates. Some cross sections were further extended by nhc to encompass the entire Bear Creek floodplain. Intermediate cross sections were added at several locations to improve the model's stability and accuracy or as necessary for computation of bridge expansion and contraction losses. These were developed by interpolating the channel portion of adjacent cross sections and extending the overbanks based on the topographic base map.

Simulated water-surface elevations, field reconnaissance, and anecdotal reports from residents indicate that during severe floods, flow breaks out of the main Bear Creek channel downstream of NE 95th Street and passes to the east of the Firneldy Village mobile-home park. This split flow travels overland in a southerly direction, joins floodwater from Evans Creek, and returns to the Bear Creek system near the confluence of these creeks. The split flow was modeled in the HEC-2 model using the weir split-flow option. The split flow returns to Bear Creek via the Evans Creek overbank so the modeled Evans Creek discharges were modified to reflect this additional flow.

Because the Bear Creek split flow affects water-surface elevations in Evans Creek, and the two creeks jointly share an extensive floodplain at their confluence, the restudy included detailed hydraulic modeling of Evans Creek from its mouth at Bear Creek upstream to River Mile 0.74.

Roughness values (Manning's "n") used in the HEC-2 model were determined by calibrating the Bear Creek model to the January 18, 1986, and January 10, 1990, flood events. High-water data for these events were obtained from various sources, including a report by CH2M HILL for the City of Redmond (Reference 111); a hydraulic analysis by CH2M HILL for the WSDOT (Reference 112); and photographs by the City of Redmond, the owners of Friendly Village, and the owners of the Redmond Animal Clinic. Anecdotal reports of flooding were also provided by the owner of the farm near the confluence of Bear and Evans Creeks and the owners of property near the NE 106th and NE 116th Street crossings. These events are the most significant floods recorded in recent history and provide useful data for calibration of roughness coefficients both in the channel and on the overbank floodplain. Most of the calibration data are for the reach of Bear Creek downstream of NE 95th Street. For other reaches of the creek for which little or no calibration data were available, roughness coefficients were estimated using engineering judgment and reference to classical publications (References 113 and 114). Manning's "n" values range from 0.045 to 0.075 for the main channel and from 0.050 to 0.200 for the overbank and floodplain.

Starting water-surface elevations at the downstream end of the Bear Creek restudy reach were extracted from the most recent approved LOMR for lower Bear Creek by the Montgomery Water Group, Inc., (References 115-117).

The 1987 aerial photogrammetry and base maps show that the restudy reach of Bear Creek (between State Route 202 and the uppermost Avondale Road crossing) is approximately 0.4 mile longer than that shown in the previous study profiles. This could be the result of changes in the stream channel but is most likely a result of improved photogrammetric techniques. The revised profile panels are measured in feet above State Route 202 along the restudied portion of Bear Creek.

The floodplain boundaries for the 1-, and 0.2-percent-annual-chance events were taken from a topographic work map at a scale of 1:2,400. The base map was obtained from the KCSWM and was prepared by David Smith and Associates from aerial photographs taken in March 1987.

The floodway boundaries developed in the restudy were computed with the HEC-2 model, generally with the assumption of equal-conveyance reduction from each side of the floodplain (HEC-2 encroachment method 4). The floodway model run was complicated by several factors. First, subsequent to the preparation of the previous study, several large fills were placed in the floodway fringe, thus using a portion of the allowable floodway surcharge. These fills include a large fill on the left bank downstream of Union Hill Road, a fill on the right bank between Union Hill Road and the Avondale Road Extension, the roadway fill of the Avondale Road Extension, and a large fill on the left bank upstream of the Avondale Road Extension to the north side of Union Hill Road. Similarly, several bridges have been replaced with large structures subsequent to the previous hydraulic analysis, tending to lower water-surface elevations for the same discharges. Based on NFIP regulations, target water-surface elevations for the floodway runs were based on a 1 foot surcharge above baseline conditions at the time of the previous study of 1978.

The second factor complicating the floodway analysis is that the current hydraulic modeling shows significant deviations from the computed 1-percent-annual-chance water-surface elevations reported in the previous study, particularly in the reach below the confluence with Evans Creek. Some of the differences result from the floodplain modifications described above. However, further investigation showed that the greatest portion of the difference is a result of the selection and application of the hydraulic model. The previous study analysis was performed with the USACE, Seattle District, step-backwater model (1983) using a total of six channel cross sections and one bridge to define the reach between State Route 202 and the confluence of Bear and Evans Creeks. In contrast, the restudy uses the USACE HEC-2 model and a total of 30 channel cross sections and four bridges in this reach.

A third factor complicating the floodway analysis was that HEC-2 is unable to use the split-flow option and automatic floodway encroachment options together. This necessitated the construction of a model of the existing condition with the split flow removed (a pseudo 1-percent-annual-chance flood model) as the basis for the floodway runs. Finally, although the automated encroachment option in HEC-2 is designed to meet target water-surface elevations at each cross section, there are cases where the model does not limit the surcharge to the desired elevation or results in an unusual floodway shape. Therefore, the floodway model runs were performed in the following manner:

- A baseline HEC-2 model was configured corresponding to the 1978 conditions using recent channel survey data with the overbanks modified to remove fills and bridge modifications that have occurred since 1978. This model was run to determine appropriate regulatory BFEs.

- Target floodway elevations were computed as the regulatory BFEs plus 1 foot.

- A floodway HEC-2 model was configured to reproduce results of the existing condition 1-percent-annual-chance profile while eliminating the split-flow cards. This model was run using only the flow in the main channel (minus the portion that had previously been computed as split flow) to develop a pseudo 1-percent-annual-chance profile that provided HEC-2 with a basis for the automatic encroachment run.

- A second profile was run using the floodway model with the full 1-percent-annual-chance discharge and the equal-conveyance reduction encroachment option (HEC-2 method 4). Target surcharges as established using the 1978 baseline model were input for this model run.

- The floodway model was revised iteratively using manual encroachments (HEC-2 method 1) to meet surcharge targets (regulatory BFEs plus 1 foot) and provide a reasonable shaped floodway.

Using the final HEC-2 floodway model, floodway widths were computed at each cross section. Between cross section, the floodway boundaries were interpolated. As a result of the restudy,

Table 6, “Floodway Data,” was revised. The “Regulatory” and “Without Floodway” elevations are based on existing conditions. The surcharge is the difference between the existing “With Floodway” elevation and the 1-percent-annual-chance water-surface elevation using the 1978 baseline model. Flood Profile Panels for Bear and Evans Creeks were revised as a result of the restudy.

South Fork Skykomish River - The cross-section data for the study along the South Fork Skykomish River was taken from field surveys and topographic mapping prepared by David C. Smith and Associates, Inc. The water-surface elevations of the floods of the selected recurrence intervals were computed using HEC-2. The 1-percent-annual-chance floodplain boundary was delineated using water-surface elevation determined at each cross section. Between cross sections, the 1-percent-annual-chance floodplain was interpolated using topographic mapping at a scale of 1:2,400, with contour intervals of four feet.

Channel and overbank roughness factors (Manning’s “n”) used in the hydraulic analyses were based on engineering judgment. The range of channel roughness factors of 0.038 to 0.048 and overbank roughness factors of 0.080 to 0.120 were used to model the South Fork Skykomish River.

Middle Fork Snoqualmie River - The cross-section data for the study along the Middle Fork Snoqualmie was taken from field surveys and topographic mapping prepared by David C. Smith and Associates, Inc. The water-surface elevations of the floods of the selected recurrence intervals were computed using HEC-2. The 1-percent annual chance floodplain boundary was delineated using water-surface elevation determined at each cross section. Between cross sections, the 1-percent-annual-chance floodplain was interpolated using topographic mapping at a scale of 1:2,400, with contour intervals of 2 and 10 feet. Flood profiles for the Middle Fork Snoqualmie River were calibrated using high-water marks at the Mount Si Road bridge.

Channel and overbank roughness factors (Manning’s “n”) used in the hydraulic analyses were based on engineering judgment. The hydraulic profile for the Middle Fork Snoqualmie River was generally calibrated to a known flood-stage water-surface elevation (at the bridge where a high-water mark was identified). The estimated roughness coefficients for this study were adjusted to attain a relatively close elevation match to known high-water marks.

North Fork Snoqualmie River - The cross-section data for the study along the North Fork Snoqualmie Rivers was taken from field surveys and topographic mapping prepared by David C. Smith and Associates, Inc. The water-surface elevations of the floods of the selected recurrence intervals were computed using HEC-2. The 1-percent-annual-chance floodplain boundary was delineated using water-surface elevation determined at each cross section. Between cross sections, the 1-percent-annual-chance floodplain was interpolated using topographic mapping at a scale of 1:2,400, with contour intervals of 2 and 10 feet.

The range of channel roughness factors of 0.035 to 0.046 and overbank roughness factors of 0.070 to 0.100 were used to model the North Fork Snoqualmie River.

The floodway was determined based on equal-conveyance reduction from both sides of the floodplain. Floodway widths were determined at each cross section, and between cross sections the floodway boundaries were interpolated. In cases where the floodway line is collinear with the 1-percent-annual-chance floodplain line, only the floodway line has been shown.

3.2.5 Revision 5

North Creek - The hydraulic analyses for the revised study were performed using the USACE HEC-2 computer program (Reference 97). The physical geometry of the North Creek channel was represented by 39 cross sections surveyed by nhc between December 1993 and February 1994. Only the channel portion of each section was surveyed. The cross sections were extended to include the floodplain using 2-foot-contour-interval mapping provided by the City of Bothell Department of Public Works (Reference 120) and the Quadrant Company. The HEC-2 model contains the surveyed sections as well as sections synthesized from the survey data to define the characteristics of bridges and complex study areas.

The starting water surface elevations were determined from the flood profiles computed for the original study for the 10-, 2-, and 1-percent-annual-chance events. The 0.2-percent-annual-chance flood profile was not computed for the previous study due to complex hydraulic conditions downstream of the County line. Therefore, the starting water-surface elevation for the 0.2-percent-annual-chance event was determined based on normal depth.

Channel roughness coefficients (Manning's "n" values) used in the HEC-2 model were determined by calibrating the model to conditions observed in the field on December 10, 1993. The December 10 calibration event generally stayed within the channel banks. Therefore, floodplain "n" values were estimated using engineering judgment and reference to classical publications (References 98 and 99). The final calibrated "n" values for North Creek are shown in Table 4, "Manning's "n" Values."

Twelve bridges are represented in the HEC-2 model for the revised reach of North Creek. The data used to define these structures were obtained during nhc field surveys. No other permanent structures were identified that would significantly affect flood levels.

Downstream of the King-Snohomish County line, North Creek is confined between levees. At the County line, tieback levees have been constructed across both the left and right floodplains to direct upstream flow into the North Creek channel. Just upstream of the County line, in the Monte Villa Center development, a setback levee parallels the channel to the east. At the County line, it connects to the downstream levee. At its upstream end, it tapers into higher ground near 240th Street Southeast.

The 1-, and 0.2-percent-annual-chance floodplain boundaries were delineated using the flood elevations determined at each cross section. Between cross sections, the boundaries were interpolated using topographic maps at a scale of 1"=200', with a contour interval of 2 feet (Reference 121).

North Creek (LOMR) - The base condition HEC-2 hydraulic model (Reference 97) for North Creek was revised to reflect the levee system and new topographic information. The use of a revised base condition hydraulic model resulted in both increases and decreases in the BFEs along the revised reach of North Creek within the levee system. The BFEs decreased by 0.2 foot to 0.3 foot from approximately 400 feet upstream of I-405 to just downstream of the southernmost North Creek Parkway bridge crossing, and increased by 0.3 feet to 1.4 feet from approximately 500 feet upstream of the southernmost North Creek Parkway bridge crossing to just upstream of the northernmost North Creek Parkway bridge crossing.

The Special Flood Hazard Area (SHFA) is contained by the levee system along this reach of North Creek and, therefore, the SFHA width decreased and the areas protected form 1-percent-annual-chance flooding by the levee system have been redesigned Zone X.

The floodway for the reach of North Creek from I-405 to 240th Street Southeast was computed based on incorporating the credited levee system and equal conveyance reduction from each side of the flooding.

3.2.6 Revision 6

Tolt River - The hydraulic analysis was performed by Harper Righellis Inc. using the USACE HEC-2 step backwater computer program (Reference 97). Data for the cross sections were taken from field surveys performed in August through November, 1994 and from data extracted from planimetric maps. The starting water-surface elevation was obtained by the slope-area method based on an estimated slope of the energy grade. The roughness coefficients were adjusted to calibrate the hydraulic model to observed high water marks, and the range of values is shown in "Manning's "n" Values", Table 4.

From just upstream of the abandoned railroad (Snoqualmie Valley Trail) to the Holberg levee area, Tolt River is confined between levees. However, these levees do not meet FEMA freeboard requirements. Therefore, the water-surface profiles for the area affected by the levees are computed for both with and without consideration of the levees.

The 1-percent-annual-chance floodplain boundaries for Tolt River were delineated using the flood elevations determined at each cross section. Between cross sections, the boundaries were interpolated using topographic maps at a scale of 1:2,400, with a contour interval of 2 feet (Reference 123).

South Fork Snoqualmie River – The study reach is called the Upper South Fork Snoqualmie River for the purposes of this study to better describe the affected flooding area.

Topographic maps from studies completed by Harper Righellis, Inc. for the South Fork were used for this restudy (Reference 126).

Cross sections for the mainstem were converted from an HEC-2 data deck from a study currently underway by the USACE (Reference 127). Overbank portions of some of these cross sections were modified using the new topographic maps as produced by Harper Righellis Inc. Cross sections for the Middle

Fork and the South Fork upstream of I-90 were converted from the HEC-2 data deck from a study recently completed by Harper Righellis Inc. (Reference 128).

3.2.7 Revision 7

Snoqualmie River - Analyses of the hydraulic characteristics of flooding from the studied sources were performed to provide estimates of the elevations of floods of the 10-, 2-, 1-, and 0.2-percent-annual-chance recurrence intervals. Users should be aware that flood elevations shown on the Digital Flood Insurance Rate Map (DFIRM) represent rounded whole-foot elevations and may not exactly reflect the elevations shown on the Flood Profiles (Exhibit 1) or in the Floodway Data table in the FIS report. Flood elevations shown on the DFIRM are primarily intended for flood insurance rating purposes. For construction and/or floodplain management purposes, users are cautioned to use the flood elevation data presented in this FIS in conjunction with the data shown on the DFIRM.

The prior USACE hydraulic analyses were reviewed in detail, and appropriate revisions were made. The revisions include updating some cross sections based on more recent channel surveys and modifying the effective limits of flow, roughness coefficients, expansion and contraction coefficients, peak flows, and starting condition methods.

Water surface elevations (WSELs) for the 1-percent-annual-chance flood on the Snoqualmie River, Ribary Creek, and Gardiner Creek were computed using the USACE Hydrologic Engineering Center River Analysis System (HEC-RAS Version 2.2 Reference 131), step-backwater computer program.

Because the Middle Fork and South Fork peak flows are near coincident, all the hydraulic analysis models assume coincident peak flows; therefore, the starting condition for each model is the WSEL of the appropriate cross section of the downstream model. The main stem model starting WSEL was taken from the FEMA published WSELs. The overflow values from Middle Fork to South Fork were estimated using engineering judgment based on the terrain, because cross sections were not available at the split location to yield a more precise computation. The Gardiner Creek and Ribary Creek starting WSELs were based on a known WSEL at the downstream end.

Roughness coefficients (Manning's "n") values for South Fork Snoqualmie River, Snoqualmie River main stem, Middle Fork Snoqualmie River-Overflows, Gardiner Creek, and Ribary Creek are shown in Table 4.

Ribary Creek detailed study elevations were superseded by the elevations of South Fork using the "without levee" analysis. The floodplain delineation at the confluence of Gardiner Creek with South Fork was based on the South Fork model.

Because the levees on South Fork, beginning at the I-90 bridge and extending downstream to the Snoqualmie Valley Trailbridge, did not meets FEMA's standards for providing protection from the 1-percent-annual-chance flood, "with levee" and "without levee" conditions were analyzed. To reflect the levees on both sides of the river, the following analyses were conducted: "with both levees", "without right levee", "without left levee."

The regulatory floodway along the Snoqualmie River study reach was determined using the equal-conveyance reduction option in the HEC-RAS backwater model from each side of the floodplain.

The Floodway Data Table and the FIRM show the results of the floodway computation for the studied reach of the Snoqualmie River.

The boundaries of the area inundated by the 1-percent-annual-chance flood were plotted on USGS 1:24,000-scale Digital Raster Graphic (DRGs) enlarged to 1:2,400 (Reference 132). Topographic data, roads, and canals on the DRGs; recent aerial photographs; and field observations were reviewed to aid in plotting the flood boundaries between cross sections. Inundated areas with little or no flow were identified. More precise data on the extent of inundation may be determined at any given location by using the computed WSEL and detailed field surveys of the land surface.

Issaquah Creek - Analyses of the hydraulic characteristics of flooding from the sources studied were performed to provide estimates of the elevation of floods of the selected recurrence intervals. Users should be aware that flood elevations shown on the DFIRM represent rounded whole-foot elevations and may not exactly reflect the elevations shown on the Flood Profiles or in the Floodway Data tables in the FIS report. Flood elevations shown on the DFIRM are primarily intended for flood insurance rating purposes. For construction and/or floodplain management purposes, users are cautioned to use the flood elevation data presented in this FIS in conjunction with the data shown on the DFIRM.

Cross-section and bridge data for the backwater analysis on Issaquah Creek and East Fork were field surveyed in April and May 2000 and February 2001 to obtain invert elevations and other hydraulic parameters. To define overbank areas and areas in-between cross sections, these data were supplemented with City of Issaquah digital mapping with a contour interval of 2 feet from Nies based on a March 1988 aerial survey. High-water mark data based on community input were also field surveyed as part of this study.

WSELs of floods of the selected recurrence intervals on Issaquah Creek, East Fork, and Gilman Boulevard Overflow were computed using the USACE HEC-RAS, Version 3.0.1, step-backwater computer program (Reference 140). The hydraulic analyses for this study were based on unobstructed flow. Therefore, the flood elevations shown on the profiles are considered valid only if hydraulic structures remain unobstructed, are operated properly, and do not fail.

All elevations are referenced to North American Vertical Datum of 1988 (NAVD). Refer to section 3.3 Vertical Datum for more information. To obtain up-to-date elevation information on NGS ERMs shown on the DFIRM, please contact the Information Services Branch of the NGS at (301) 713-3242 or visit their website at www.ngs.noaa.gov. Map users should seek verification of non-NGS ERM monument elevations when using these elevations for construction or floodplain management purposes.

The starting WSELs on Issaquah Creek at the northern corporate limit of the City of Issaquah were based on previous studies. The water surface elevations published in the King County FIS closely matched the predicted elevations for this analysis at that location.

The starting WSELs on East Fork were developed through normal depth computation using the slope-are method. The regulatory WSELs were influenced by backwater from the main stem of Issaquah Creek, as shown on the Flood Profiles.

The starting WSELs of floods of the selected recurrence intervals on the Gilman Boulevard Overflow and the main stem of Issaquah Creek were set using computed WSELs at hydraulic control sections. The upper main stem starting WSEL was set at the upper fish hatchery weir control section. The Gilman Boulevard Overflow model starting WSEL was set below a culvert control section.

Channel and overbank roughness factors (Manning's "n" Values) used in the hydraulic computations were chosen by engineering judgment and were based on field observations of the stream and floodplain areas and on hydraulic calibration of flood profiles to available high-water mark data. The February 8, 1996, flood event was used for hydraulic model calibration. Model calibration results are discussed in detail in the calibration and bridge improvement memorandum by the Montgomery Water Group for Issaquah Creek, East Fork, and the Gilman Boulevard Overflow path are listed in Table 4.

Locations of selected cross section used in the hydraulic analyses are shown on the Flood Profiles. For stream segments for which a regulatory floodway was computed (see Section 4.2), selected cross-section locations are also shown on the DFIRM.

The NFIP encourages State and local governments to adopt sound floodplain management programs. To assist in this endeavor, each FIS provides 1-percent-annual-chance floodplain data, which may include a combination of the following: 10-, 2-, 1-, and 0.2-percent-annual-chance flood elevations; delineations of the 1-, and 0.2-percent-annual-chance floodplains; and the 1-percent-annual-chance floodway. This information is presented on the DFIRM and in many components of the FIS, including Flood Profiles, Floodway Data tables, and the Summary of Discharges table. Users should reference the data presented in the FIS as well as additional information that may be available at the local community map repository before making flood elevation and/or floodplain boundary determinations. Overflows from Issaquah Creek and East Fork are shown on the maps as shallow flooding zone (Zone AO) with average depths identified.

To provide a national standard without regional discrimination, the 1-percent-annual-chance flood has been adopted by FEMA as the base flood for floodplain management purposes. The 0.2-percent-annual-chance flood is employed to indicate additional areas of flood risk in the community. For each stream studied by detailed methods, the 1- and 0.2-percent-annual-chance floodplain boundaries have been delineated using the flood elevations determined at each cross section. Between cross sections, the boundaries were interpolated, using digital topographic maps with contour intervals of 2 feet (Reference 142).

The 1- and 0.2-percent-annual-chance floodplain boundaries are shown on the DFIRM. On this map, the 1-percent-annual-chance floodplain boundary corresponds to the boundary of the areas of special flood hazards (Zones AE, AH, and AO), and the 0.2-percent-annual-chance floodplain boundary corresponds to the boundary of areas of moderate flood hazards. In cases where the 1- and 0.2-percent-annual-chance floodplain boundaries are close together, only the 1-percent-annual-chance floodplain boundaries may lie above the flood elevations but cannot be shown because of limitations of the map scale and/or lack of detailed topographic data.

Encroachment on floodplains, such as structures and fill, reduces flood-carrying capacity, increases flood heights and velocities, and increases flood hazards in areas beyond the encroachment itself. One aspect of floodplain management involves balancing the economic gain from floodplain development against the resulting increase in flood hazard. For purposes of the NFIP, a regulatory floodway is used as a tool to assist local communities in this aspect of floodplain management. Under this concept, the area of the 1-percent-annual-chance floodplain is divided into a floodway and a floodway fringe. The floodway is the channel of stream, plus any adjacent floodplain areas, that must be kept free of encroachment so that the 1-percent-annual-chance flood can be carried without substantial increases in flood heights. Minimum Federal standards limit such increases to 1 foot, provided that hazardous velocities are not produced. The floodways in this study are presented to local agencies as a minimum basis for additional floodway studies.

The floodways presented in this study were computed for certain stream segments on the basis of equal conveyance reduction from each side of the floodplain. Floodway widths were computed at cross sections. Between cross sections, the floodway boundaries were interpolated. The results of the floodway computations are tabulated at selected cross sections. In cases where the floodway and 1-percent-annual-chance floodplain boundaries are either close together or collinear, only the floodway boundary has been shown.

The area between the floodway and 1-percent-annual-chance floodplain boundaries is termed the floodway fringe. The floodway fringe encompasses the portion of the floodplain that could be completely obstructed without increasing the water-surface elevations of the 1-percent-annual-chance flood more than 1 foot at any point. The Flood Profiles, Floodway Data tables, and the DFIRM show the results of the floodplain and floodway computations for the studied reaches of Issaquah Creek, including East Fork. Floodways were not computed for the Gillman Boulevard Overflow. The Gillman Boulevard Overflow area is designated on the DFIRM as a breakout flow area, where the flow conveyance during the base flood must be maintained to avoid increasing downstream flood hazards in Issaquah Creek. This breakout flow area extends from the left overbank (looking downstream) of Issaquah Creek between Cross Sections M and N toward the west along Gillman Boulevard.

3.2.8 Revision 8

Cedar River – King County Unincorporated Area

The river was modeled using HEC-RAS 2.2. The computations are based on sub-critical flow and use the slope (normal depth) method for the starting condition. The HEC-RAS model conforms to the criteria of hydraulic modeling. There are a few locations where water-surface elevations are higher than the end points of cross sections; but in all cases, the end points are at the effective flow limit for that cross section. Aside from directly within some bridges, negative floodway surcharges are limited in location and are negligible (i.e., all are much less than -0.1 feet).

Cedar River –City of Renton Area - Detailed methods were used to define the hydraulic characteristics of the 5.36-mile study reach. A HEC-RAS model was previously created for the main channel and overbank floodplain of the Cedar River from Lake Washington upstream to 149th Avenue Southeast (also referred to as Jones Road or the Elliot bridge). During this current study, the model was modified to include four additional split flow reaches and updated survey data at select cross sections. The first of these split flow reaches occurs at Maplewood Golf Course (about river mile 4.5). This reach, designated as the “Golf Course Split”, defines the flow path where a portion of the 0.2-percent-annual-chance flood flow leaves the main channel and travels overland through the golf course before rejoining the main channel at approximately river mile 2.8. A second split flow reach, termed the “Maplewood Overflow”, routes floodwaters through a portion of the Maplewood subdivision. Another split flow reach, designated as the “Old Channel Split”, was defined for the portion of the old channel that was cut-off as a result of the landslide. The fourth and final split flow reach occurs south of Highway 169 between river miles 4.2 and 5.36. Floodwaters enter this reach, designated as the “Highway 169 Overtopping Split”, between river miles 5.0 and 5.3

and are prevented from rejoining the main channel again until river mile 4.5 (approximately 140th Ave. SE).

One of the most notable features of this model is that the channel geometry used to determine flood risk represents the maximum “allowable” bed elevation prior to mandatory dredging of the lower river as detailed in the Cedar River at Renton Flood Damage Reduction Project Operation and Maintenance Manual (O&M Manual) (Reference 144). This future aggraded condition depicts a significantly higher channel bed profile than existed at the time of the channel surveys for this study. Because the maximum “allowable” bed profile defines the highest possible bed profile allowed in the O&M Manual, FEMA requires that it must be considered when determining flood risk. The difference in the allowable bed and surveyed channel bed profiles is illustrated in Exhibit 1.

The HEC-RAS model was used to compute water surface profiles for the 10-, 2-, 1- and 0.2-percent-annual-chance flood events, floodplain inundation limits for the 1-, and 0.2-percent-annual-chance events, and floodway boundaries for the 1-percent-annual-chance flood.

Fifteen bridges influence hydraulic conditions within the study reach. Three other bridges completely span the river (the two I-405 bridges and the Burlington Northern railroad bridge just downstream of I-405), and one other bridge is hydraulically lifted above the water surface during large flood events (the south Boeing bridge at river mile 0.75). The deck of the old railroad bridge at river mile 2.9 (now a pedestrian bridge) is above both the 1-, and 0.2-percent-annual-chance flows, and therefore only the bridge piers were included in the model. This was also the case with the 149th Avenue Southeast bridge, at the upstream end of the study reach. In two locations, immediately adjacent parallel bridges were modeled as a single bridge. This was done at the Houser Way and downstream pedestrian bridges, as well as at the Highway 169 and downstream pedestrian bridges. Seven of the bridges experience partial or complete pressure flow during the 1-percent-annual-chance flood and eight at the 0.2-percent-annual-chance flood. Sensitivity analysis showed that three of these bridges were best modeled using the pressure flow option for high flows while the remainder was more accurately simulated using the energy method for high flows. The three bridges modeled with the pressure flow option were: the pedestrian bridge under I-405, the Houser Way/pedestrian bridge, and Wells Avenue Bridge in downtown Renton.

Channel and overbank roughness coefficients (as represented by Manning's "n" values) were initially estimated from several established references (References 99 and 147). These values were further refined by calibrating the HEC-RAS model to two recent flood events, the flood of record on November 24, 1990 and a lesser event on November 30, 1995. The 1990 flood had a peak discharge of 10,600 cfs as estimated by the USGS at Gage 12119000 in downtown Renton, and the 1995 flood had a peak discharge of 7,650 cfs. Highwater marks were surveyed by the U.S. Army Corps of Engineers after both floods.

Calibration was difficult at the downstream end of the study reach because the channel and overbank have changed significantly since the 1990 and 1995 events. The reach extending from Lake Washington upstream to Logan Avenue was dredged in 1986 and again in 1998, while periodic surveys showed that the channel experienced significant aggradation between dredging operations. In addition, a new floodwall-levee system was recently constructed downstream of Logan Avenue which would prevent water from leaving the main channel and flowing across the airport and Boeing property, as occurred during the 1990 event. Manning's "n" values in this reach were largely taken from a previously calibrated HEC-RAS model created by USACE to design the floodwall-levee system.

Upstream of I-405 the current model calibrated fairly well to observed highwater marks. The resultant Manning's "n" values range from 0.02 to 0.045 in the main channel, which varies from 80 to 150 feet wide and has a gradually meandering planform with occasional gravel bars. Main channel "n" values are typically 0.033 throughout most of this reach, but were raised in the vicinity of the old railroad bridge at river mile 2.9 to reflect the turbulence generated by the two sharp bends in the river and to match highwater marks surveyed just upstream. The channel banks are typically overgrown with dense vegetation such as blackberry bushes, while the floodplain varies from groomed lawn to thick brush. Manning's "n" values ranging from 0.03 to 0.15 were used on the overbank and floodplain.

Starting water surface elevations at the downstream end of the modeled reach were set at 17.06 feet (NAVD 88). Water levels in Lake Washington are regulated by the Chittenden Locks. This elevation corresponds to the maximum expected water surface elevation in Lake Washington between November 1 and March 31 (Reference 146), as well as the elevation used in the design of the flood protection project by USACE.

The USACE designed and constructed a series of floodwalls and levees along the lower end of the study reach, extending from Lake Washington to just upstream of Logan Avenue. The levees and floodwalls, in conjunction with modifications to the south Boeing Bridge and a program of dredging, were designed to provide 1-percent-annual-chance flood protection at the 90 percent reliability level. Because the project is USACE certified, the reach was modeled with the levees and floodwalls in place, without the south Boeing Bridge (which is lifted hydraulically above the water during flood events), and using an aggraded bed scenario consistent with the maximum "allowable" bed profile specified in the Cedar River at Renton Flood Damage Reduction Operation and Maintenance Manual (Reference 144).

The flood profiles for the 10-, 2-, 1-, and 0.2-percent-annual-chance floods along the main stem of the Cedar River were generated using the HEC-RAS model, and are illustrated in Exhibit 1.

The 1-percent-annual-chance floodway boundaries developed in this study were determined with the HEC-RAS model, generally with the assumption of equal conveyance reduction from each side of the floodplain (HEC-RAS method 4). At some locations, applying the automatic encroachment feature available in HEC-RAS produced flood elevation increases greater than 1 foot and resulted in an unusual floodway shape. As a result, the encroachments were manually adjusted using HEC-RAS method one until a reasonable floodway was established. Further upstream, the floodway was located at the edge of the active channel, existing wetlands, and salmon spawning channels, even though additional encroachment would be possible without causing greater than a 1 foot rise in water surface elevations. No separate floodway was computed for the split flow reaches because flow was assumed to be contained in the main channel by the floodway encroachments.

Floodway widths were computed at each cross section. Between sections, the floodway boundaries were interpolated. The results of the floodway analysis are tabulated for each cross section in Table 5. In locations where the floodway and the 1-percent-annual-chance floodplain boundary coincide, only the floodway boundary is shown.

Kelsey Creek - Analysis of the hydraulic characteristics of flooding along Kelsey Creek and the West Tributary study reaches were performed using detailed methods. Detailed methods involved using a HECRAS (USACE, 2005) water surface profile computer model for Kelsey Creek and the West Tributary developed by nhc. The following sections describe the data, information, and assumptions used to construct the hydraulic model.

Channel and Floodplain Topography

The HEC-RAS model for Kelsey Creek and the West Tributary includes 197 and 42 cross sections, respectively. These cross sections serve to represent the geometry of the channel and floodplain along the study reach. Cross-section data for this study came from two different sources. Channel cross sections along Kelsey Creek were surveyed by the City of Bellevue in early 2006, while cross sections on the West Tributary were surveyed by nhc as part of a different project for the City of Bellevue Parks Department in 2004. Topographic data consisting of two foot contour intervals, provided by the City of Bellevue, were used to extend the surveyed channel cross sections to include the floodplain.

Hydraulic Structures

The hydraulic analysis of the Kelsey Creek and West Tributary study reaches includes 16 culvert, 31 bridge, 21 inline weir, and seven lateral weir structures. Within the HEC-RAS model, all culverts were modeled using the Highest Upstream Energy method, while bridges were modeled either with the Energy, or Pressure and/or Weir methods, depending on level of inundation. The 21 inline weirs are located on the main stem of Kelsey Creek between Cross Sections K and P, and generally consist of concrete grade control structures.

Split Flow

Lateral weir structures were utilized in the HEC-RAS model to transfer flow between the main stem of Kelsey Creek and an adjacent swale-like reach located in Kelsey Creek Park immediately to the west. As previously discussed an earthen embankment was 'failed' to allow overtopping flow in the main stem to move into the swale. In this reach the current location of the creek is perched at an elevation above the adjacent swale, such that any overbank flow will be uncontained and move laterally. A split flow optimization routine was used for this analysis to balance water surface elevations in the main channel and discharges flowing into the swale reach.

Starting Water Surface Elevation

The downstream limit of the Kelsey Creek HEC-RAS model is located at the outlet of the culvert structure under I-405. Immediately downstream of this culvert outlet, the creek flows through a concrete fish ladder structure. Because the top of the structure is elevated approximately 8 feet above the receiving body of the water (Mercer Slough), and the flow is likely of mixed regime, i.e. rapidly transitions between super and subcritical, a critical flow depth was chosen as a starting water surface condition. Because the channel upstream of the 630-foot long I-405 culvert structure is deeply entrenched within a narrow gully, the floodplain extents are not significantly affected by starting water surface elevation. Thus, the choice of using the critical depth boundary condition is considered reasonable in this situation.

Model Calibration

Twelve high water marks along Kelsey Creek and the West Tributary were observed by City of Bellevue staff following the major flood event of October 20, 2003 (approximately 25-year return period). Initial channel and floodplain roughness factors (Manning's "n" values) were estimated based on field observations and engineering judgment. To calibrate the hydraulic model, these initial roughness factors were adjusted, but kept within a range of values that is consistent with past experience, until the computed water surface elevations closely matched the recorded high water mark elevations. The resulting channel and floodplain "n" values for Kelsey Creek and the West Tributary range from 0.035 to 0.06 and 0.035 to 0.15, respectively.

Flood Profiles

The 10-, 2-, 1-, and 0.2-percent-annual-chance flood profiles events for the Kelsey Creek and the West Tributary study reaches generated using the HEC-RAS model constructed for this study. These profiles represent conditions of unobstructed flow, meaning that the bridge and culvert openings as well as the main channel remain unobstructed during flood events.

Other Studies

In 2003, a LOMR (Case No. 03-10-0399P) was submitted for the reach just upstream (south) of NE 6th Street and accepted by FEMA's reviewing agents (Montgomery, 2003). The upstream most 1-percent-annual-chance BFE for the current study is estimated at 252 feet, NAVD 88 located just upstream of NE 6th Street, while the 2003 LOMR uses a starting water surface elevation of 253 feet, NAVD 88, approximately 400 feet upstream. These values are sufficiently close to tie in the floodplain hazard areas and profiles between this and the 2003 LOMR studies. It should be noted that the limit of the floodway analysis is defined at the upper most cross section of this study, as a floodway did not exist for effective Kelsey Creek FIS, or the 2003 LOMR.

Patterson Creek - A HEC-RAS computer model was created to simulate the hydraulic characteristics of the study reach. The model was used to compute water surface profiles corresponding to the 10-, 2-, 1-, and 0.2-percent-annual-chance floods, flood inundation limits for the 1-percent-annual-chance (base flood) and 0.2-percent-annual-chance events, and the floodway boundary for the 1-percent-annual-chance flood.

One hundred and five cross sections are used in the HEC-RAS model to represent the channel and floodplain geometry along the study reach. Most of these cross sections were surveyed by Minister-Glaeser Surveying (MGS) in December 2005. Additional cross sections were interpolated from the survey and topographic data where needed. The cross-section surveys typically only included the stream channel from bank to bank. The floodplain was not surveyed; therefore, the overbank portion of each cross section was added using the digital topographic data developed for this study by 3Di-West. The topographic data was created using a combination of photogrammetric techniques and LIDAR data. Aerial photographs of the study reach were taken in March 2004.

One culvert and 13 bridges influence hydraulic conditions within the study reach. The culvert is located at the upper-most crossing of the Redmond-Fall City Road. The 12 bridges included in the model are located on driveways to private residences or private and public roads; including State Highway 202. One bridge within the study reach, at NE 4th Place, was not included in the model because access to the private road was denied and therefore detailed information on the bridge was not available. All other bridges and the Redmond-Fall City Road culvert were surveyed by nhc to obtain elevation data and structural geometry for input into the hydraulic model.

The hydraulic model was extended downstream to the confluence with the Snoqualmie River to provide a more refined estimate of water levels at the downstream end of the detailed study reach. In accordance with the *Guidelines and Specifications for Flood Hazard Mapping Partners* (2003), the starting water surface elevation for the backwater model was assumed to be normal depth. The assumption of coincident peaks with the Snoqualmie River did not meet the acceptance criteria in the FEMA guidelines. Backwater flooding from the Snoqualmie River will influence the lower two miles of Patterson Creek. FEMA will use the information contained in this study and the information contained in the Snoqualmie River Floodplain Mapping Study (Section 10.8.3) to determine base flood elevations to be shown for this reach in the final FIS and mapped on the final DFIRM.

Channel and overbank roughness factors (Manning's "n" values) used in the hydraulic computations were chosen using engineering judgment and were based on field observations, orthophotos, and published data. The "n" values for the main channel of Patterson Creek range from 0.04 at the downstream end of the study reach to 0.12 in the heavily vegetated and narrow wetlands areas. Overbank "n" values range from 0.02 on the golf course to 0.08 in the thick brush of the wetlands.

There were no high water marks available to calibrate the model. The only marks available for calibration were the water surface elevations at each cross section noted by the surveyors. The discharge in the channel at the time of the survey varied from 1 to 23 cfs, depending on day and location. The model was calibrated to reproduce the observed stages, all the while keeping in mind the focus of the model is for a 1-percent-annual-chance event.

All flood insurance studies are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum used for newly created or revised studies was the National Geodetic Vertical Datum of 1929 (NGVD 29). With the finalization of the North American Vertical Datum of 1988 (NAVD 88), most studies are being prepared using NAVD 88 as the referenced vertical datum. The hydraulic analysis for Patterson Creek was conducted using the NAVD 88 vertical datum. Elevation conversion factors between the two vertical datums vary by location and can be obtained from the National Geodetic Survey's VERTCON utility (Reference 133). In general, elevations along the Patterson Creek study reach can be converted from NAVD 88 to NGVD 29 elevations by subtracting 3.58 feet. Refer to section 3.3 Vertical Datum for more information.

Users should be aware that base flood elevations shown on the work map represent rounded whole foot elevations and may not exactly reflect the elevations shown on the Flood Profiles or in the Floodway Data table. Base flood elevations shown on the work map are primarily intended for flood insurance rating purposes. For construction and/or floodplain management purposes, users are cautioned to use the flood elevation data presented in the Floodway Data table as well as the Flood Profiles in conjunction with the data illustrated on the work map.

A Regulatory Floodway was delineated for Patterson Creek using the HEC-RAS model and following the FEMA *Guidelines and Specifications for Flood Hazard Mapping Partners*. In general, the floodway was developed using Encroachment Method 4 in HEC-RAS. Method 4 automatically computes encroachment stations by targeting a predefined surcharge (1 foot) while achieving an equal loss of conveyance on each overbank, where possible. At some locations, applying the automatic encroachment feature produced flood elevation surcharges significantly greater or less than 1 foot and/or resulted in an unusual floodway shape. As a result, the encroachments were manually adjusted using HEC-RAS Method 1 until a reasonable floodway was established. At some cross sections the floodway boundary coincides with the top of the channel banks. As required by FEMA, the floodway does not encroach into the active channel.

Floodway widths were computed at each cross section. Between sections, the floodway boundary was interpolated based on topographic information and to reflect general hydraulic principles. The results of the floodway analysis are tabulated for each cross section in Table 5. The floodway boundary is also shown on the work map. In locations where the floodway and the 1-percent-annual-chance floodplain boundary coincide, only the floodway boundary is shown.

Lower Snoqualmie River - A HEC-RAS unsteady flow hydraulic model was created to simulate the hydraulic characteristics of the 49-mile study reach. The model was used to compute water surface profiles corresponding to the 10-, 2-, 1-, and 0.2-percent-annual-chance floods, floodplain inundation limits for the 1-, and

0.2-percent-annual-chance events, and floodway boundaries for the 1-percent-annual-chance flood.

All of the mainstem cross sections were surveyed in March 2004 by Minister-Glaeser Surveying using bathymetric techniques. The surveyed transects included only the wetted river channel from the water's edge, bank to bank. Topographic data for the overbank portions of each cross section was derived from digital topographic data developed by 3Di-West. The topographic data was created using a combination of photogrammetric techniques and LiDAR data. Aerial photographs of the study reach were taken in March 2004.

Six bridges have potential to significantly impact hydraulic conditions within the study reach. These include the following bridges on the Snoqualmie River: SR-202 Bridge at Fall City, Tolt Hill Road Bridge over the Snoqualmie River, NE Carnation Farm Road Bridge (Stossel Fill), Novelty Bridge (NE 124th Street), Woodinville-Duvall Road Bridge, and High Bridge (Crescent Lake Road). Bridge dimensions were obtained from as-built drawings and were supplemented with field survey by nhc as necessary.

The general approach applied in this study was to characterize the probability of flooding based on an evaluation of annual peak stages rather than annual peak flows. Because of numerous complicating factors the only reliable approach to estimate flood inundation frequency was to apply an unsteady flow hydraulic model (HEC-RAS) to estimate 10-, 2-, 1-, and 0.2-percent-annual-chance (N-year) flood profiles throughout the study reach. The following steps were executed to develop the N-year, unsteady hydraulic models:

1. Reviewed USGS gage records in the Snohomish River basin and selected 16 large historic flood events to model.
2. Developed inflow hydrographs to the unsteady HEC-RAS model for the historic events. These hydrographs utilized available 15-minute and/or hourly USGS flow data, correlation coefficients, rainfall-runoff modeling, and information about reservoir operations on the Tolt and Sultan Rivers.
3. Performed hydraulic modeling of the selected flood events, including calibration/verification to seven of these historic events, and extracted peak stages at 20 key locations throughout the study reach.
4. Estimated plotting positions associated with the 16 selected flood events.

5. Manually fit non-parametric frequency curves to the peak stages obtained from step 3 using plotting positions from step 4.
6. Used the curves developed in step 5 to provide estimates of the 10-, 2-, 1-, and 0.2-percent-annual-chance stages at each key location.
7. Developed the N-year HEC-RAS models. Used a trial-and-error method to adjust historic flood inflows so that the peak stage at all key locations match the N-year stage developed in step 6.
8. Applied N-year unsteady HEC-RAS models to estimate the 10-, 2-, 1-, and 0.2-percent-annual-chance profiles throughout the study reach.

Channel and overbank roughness factors (Manning's "n" values) used in the hydraulic computations were chosen using engineering judgment and were based on field observations, orthophotos, and published data. Within the study reach, in-channel roughness values on the Snoqualmie River from 0.03 to 0.055. Overbank roughness values range from 0.02 to 0.15.

The hydraulic model was calibrated and verified to high water marks (HWMs) and/or aerial photography from seven recorded events. The Seattle District USACE provided HWMs for the following flood events: January 5, 1969; December 3, 1975; December 26, 1980; and November 23, 1986. King County and several long time valley residents provided HWMs for the November 24, 1990 storm. King County also provided oblique aerial photos of the storms on November 24, 1990, November 29, 1995, and February 9, 1996.

A significant effort was made to match each of the high water marks through refinement of the model parameters and structure. Calibration efforts included changes to the delineations of overflow reaches, adjustment of roughness and contraction and expansion coefficients, and modifications to model inputs that govern breakout flows. In some cases, the model simulated water surfaces that were higher than reported HWMs for one event while in other events the simulations yielded lower than reported peak water surface elevations. Where conflicting information was found, an effort was made to split the difference, giving more weight to the recent and larger flood events. The final calibration/verification is felt to be adequate given the complexities of the system and the limitations of a one-dimensional hydraulic model.

In general, the estimated 1-percent-annual-chance floodplain limits within the Snoqualmie Valley extend from the west valley wall to the east valley wall. There are two exceptions to this generalization. The first occurs in the reach between Snoqualmie Falls and Fall City, where the Snoqualmie River channel slope is its steepest and the mapped floodplain does not extend all the way to the east valley wall. The second exception is in the vicinity of Carnation, where the Tolt River alluvial fan has raised the valley topography and Snoqualmie River flood waters do not reach the east valley wall. The flattest portion of the flood profile on the lower Snoqualmie River occurs between the High Bridge (Crescent Lake Road) and the Woodinville-Duvall Road Bridge. The 1-percent-annual-chance water surface rises less than 2 feet across this span of 7 RM.

All flood insurance studies are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum used for newly created or revised studies was the National Geodetic Vertical Datum of 1929 (NGVD 29). With the finalization of the North American Vertical Datum of 1988 (NAVD 88), most studies are being prepared using NAVD 88 as the referenced vertical datum. The hydraulic analysis for the Snoqualmie River was conducted using the NAVD 88 vertical datum. Elevation conversion factors between the two vertical datums vary by location and can be obtained from the National Geodetic Survey's VERTCON utility (Reference 133). Refer to section 3.3 Vertical Datum for more information.

A Regulatory Floodway was delineated for the Lower Snoqualmie River using the unsteady HEC-RAS model and following the FEMA Guidelines and Specifications for Flood Hazard Mapping Partners. The hydraulic model for the baseline floodplain included eight distinct secondary flow branches in addition to the main channel reaches on the Snoqualmie River. These secondary flow branches were added to improve the model's simulation of complex floodplain hydraulic conditions including breakout flows, topographic divides, overflow channels, and storage areas. For the floodway analysis, the baseline model was modified to reflect floodplain encroachments as could be made while maintaining a flow corridor that could pass the 1-percent-annual-chance exceedence event without exceeding a 1.0 foot surcharge at any point in the main channel. The process of developing the floodway model comprised the following steps:

1. Begin with the 1-percent-annual-chance exceedence event (base flood) floodplain model.
2. Transfer the floodway limits from the effective FIS to the new hydraulic model.
3. Evaluate the surcharge of the effective floodway encroachments on water surface elevations in the new model. Like the base flood model, the floodway model is run using unsteady HEC-RAS. Thus the surcharge reflects both a loss of conveyance capacity and a reduction in flood storage.
4. Make adjustments to the effective floodplain encroachments to the extent necessary to pass the base flood without exceeding a 1-foot surcharge at any point in the main channel. To the extent possible, encroachment adjustments were made to provide an equal conveyance reduction on the left and right overbanks.
5. The modeled floodway encroachments at each cross section were plotted on the project work maps and floodway encroachments were adjusted to provide a smooth transitioning floodway delineation and to account for any areas of high ground between model cross sections.
6. The adjusted floodway encroachments from Step 5 were then reinserted in the HEC-RAS model and final floodway simulations were conducted to ensure that the surcharge criteria for the main channel were achieved.

As noted, the floodway analysis conducted by nhc focused on achieving a 1-foot surcharge in the main channel. It should be noted that there are areas where the newly estimated Base Flood Elevations (BFEs) in the overbank are not at the same level as the newly estimated main channel BFEs on the adjacent reach. This is because discharge to overflow reaches is affected by hydraulic controls in the floodplain, such as roads or high ground. Comparing the base flood elevations for the main channel with the BFEs in the adjacent overflow reaches shows that elevation differences of greater than 1 foot occur in several locations, including along the overflow branch north of Carnation, and the overflow branch east of Fall City. In these locations, and throughout the study area, the analysis focused on maintaining floodway surcharges in the main channel within the allowable 1-foot limit.

The extents of the floodway were extracted from the final floodway model at each modeled cross section. Between sections, the floodway boundary was interpolated based on topographic information and to reflect general hydraulic principles.

3.2.8.1 Springbrook Creek - Springbrook Creek was modeled using the one-dimensional unsteady flow computer program Full Equations (FEQ) developed by Delbert Franz. FEQ simulates the complex hydraulics of the Springbrook Creek system by solving the full energy equation plus continuity integrated in both time and distance along the channel. The program separates flow into two broad classes: (1) stream reaches (branches), (2) level-pool reservoirs. These two parts are then combined using different control structures, such as junctions, bridges, culverts, weirs and others. The hydraulic characteristics of branches, level-pool reservoirs, and control structures are stored in function tables. The function tables are generally computed by using FEQUTL. FEQUTL is a utility program that aids in developing input into FEQ.

The FEQ model was developed referencing NGVD 1929 vertical datum. The City's recent topographic mapping (1999) is in NAVD 1988 vertical datum. The floodplain mapping done as part of this study is all in NAVD 1988 vertical datum. Because it would require extensive effort to change the datum in the original FEQ model, it was decided to continue all hydraulic modeling in NGVD 1929 and use a datum conversion for the floodplain mapping. To convert elevations to NAVD 1988 vertical datum, 3.54 feet must be added. Refer to Section 3.3, Vertical Datum, for more information.

The model was originally developed in 1996, but was updated in 2000 and 2001 to account for changes in infrastructure and to include newly surveyed cross sections between SW 16th Street and 27th Avenue SW. These changes are documented in *Springbrook Creek Channel and Habitat Improvement Project Technical Memorandum Hydraulic Analysis* and *Springbrook Creek Channel and Habitat Improvement Project Technical Memorandum Hydraulic Analysis – Supplemental*.

In addition to the updates in 2000 and 2001, the model was updated to reflect another recent improvement. This improvement included the removal of a berm between two wetlands that were previously connected by a culvert.

Due to the two possible operation scenarios at the Black River Pumping Station (BRPS), two different simulation scenarios were developed for the 2-, and 1-percent-annual-chance events. One of the scenarios, referred to as the conveyance scenario, reflects a severe local event without BRPS pumping restrictions. This simulation assesses the conveyance capacity of Springbrook Creek. The other scenario, referred to as the storage scenario, reflects a severe Green River flood that causes the pump station to restrict its pumping rate. The storage scenario assesses the BRPS forebays as well as Springbrook Creek's and its associated wetlands' ability to temporarily contain the flood waters when the pump station discharge capacity is restricted. The higher of the resulting water levels for the two scenarios was used to determine the flood profile for the various frequency events.

Only the conveyance scenario was used for the 10-percent-annual-chance event. The storage scenario was not a concern because it was determined via frequency analyses of peak annual storage volumes in the BRPS forebay that this event would create only a negligible build-up of storage in the forebay during high Green River flows.

As noted above, the hydraulic model was originally developed in 1996. Cross-section data in the model came from a variety of sources including field survey by NRCS (formerly SCS) in 1990, more recent survey by W&H Pacific and R.W. Beck, channel design drawings, and 1980 USACE topography. It is understood that many of the data sources (i.e., cross sections) were not "as-built". In order to confirm that cross sections used in the model generally agree with the existing channel conditions, a validation or comparison was performed. Six channel cross sections were surveyed and compared to the cross sections used in the FEQ model in order to confirm use of the cross sections in the model is reasonable.

It was recognized that some differences between the modeled cross section and new surveyed cross sections would be likely. However, upon comparing the newly surveyed cross sections and the cross section used in the 1996 model, the extent of the differences, particularly downstream of SW 16th Street is fairly significant.

Due to the differences in the surveyed and modeled cross section downstream of SW 16th Street, a sensitivity analysis was performed using an existing HEC-RAS model of the area which uses the same cross sections as the FEQ model. The sensitivity analysis was performed to see if the differences in the cross sections had a significant effect on the water surface profiles. The results of the sensitivity analysis showed that using the new surveyed cross sections increased water surface elevations by more than 1 foot in some locations. The higher elevation continued upstream, although the increase in elevation decreases as you move upstream. This difference in simulated water levels is greater than the desirable accuracy of the modeling and therefore a new survey was conducted in between the upstream end of the forebay to SW 16th Street and the newly surveyed cross sections were used to modify the FEQ model for this floodplain mapping study.

Several of the Springbrook Creek valley wetlands were modeled as part of the Springbrook Creek channel. Other wetlands were modeled as level pool reservoirs because they are separated from the creek, but are connected via a pipe system or by overbank flow conditions during high flows. The storage data (stage-area relationship) for the wetlands modeled as level pool reservoirs was based on 1980 Corps of Engineers topographic mapping.

The north series of ponds is connected to Springbrook Creek with a flap gate to reduce the potential for Springbrook Creek flows to back up into the pond system. For the determination of the floodplain, it was assumed that the water levels in these ponds would match the base flood elevation of Springbrook Creek where they connect to the creek.

The floodway boundary developed in this study was determined by modeling scenarios that included filling in the floodplain (overbank areas on both sides of the channel and wetland areas) such that it causes no more than a 1-foot rise in the water surface profile. The floodway as established by the existing FIRM was used as an initial trail floodway for this study. The result was that simulated water surface elevations along the creek were well above the 1-foot rise threshold.

The hydraulic analysis used the flood events identified by the hydrologic analysis to analyze the Springbrook Creek system. An unsteady flow model was used to determine the base flood profile and the floodplain and floodway delineation. An unsteady flow model can simulate flood routing on the creek system more accurately than a steady state model because it accounts for the attenuation that occurs due to storage in the system. The Springbrook Creek system has a significant amount of storage due to adjacent wetlands, so using a model that has the ability to attenuate the peak flow through this storage was important in order to provide an accurate assessment of the water surface elevations along the creek. The hydraulic analysis was conducted using flow events as defined in HASC.

The water surface elevations in the study reach are impacted by flood flows, storage capacity of the adjacent wetlands, the conveyance capacity of the multiple culverts and bridges and the operation of the BRPS. The operation of the BRPS depends on the flow in the Green River. An unsteady (hydrodynamic) hydraulic modeling was used to characterize water surface profiles in Springbrook Creek in order to account for dynamic flood storage in study reach wetlands and, more significantly, to accurately simulate flood discharges from Springbrook Creek to the Green River via the BRPS. Pump Station operations, including limitations on pumped discharges when the Green River flows are high, impose a dynamic downstream boundary condition of the Springbrook Creek drainage system.

3.2.8.2 Green River (lower Green River) - A steady-state HEC-RAS computer model was created to simulate the hydraulic characteristics of the study reach under the condition that all levees remained in place (i.e. the current channel geometry). The model was used to compute water surface profiles corresponding to the 10-, 2-, 1-, and 0.2-percent-annual-chance floods, in-channel flood inundation limits for the 1-percent-annual-chance (base flood) and 0.2-percent-annual-chance events, and floodway data for the in-channel portion of the floodway and several isolated levee failure scenarios. A second numerical hydraulic model was developed using FLO-2D to simulate the effects of complex levee failure scenarios through the leveed portion of study reach (RM 10.6 to RM 29.7) and to define the floodplain limits and floodway path through the overbank areas. The following sections provide detailed descriptions of the development and application of the hydraulic models used in this study.

HEC-RAS Channel and Topography

Two hundred and twenty-three cross sections are used in the HEC-RAS model to represent the channel and bank geometry along the Lower Green River. The in-water portions of most of these cross sections were surveyed by Minister-Glaeser Surveying (MGS) in February 2006 with additional survey work taking place in early 2007. The in-channel cross section surveys typically only included the stream channel from edge of water to edge of water. The levee slopes and channel banks were not field surveyed; therefore, the upland portion of each cross section was added using the digital topographic data developed for this study by 3DI- West. The topographic data was created using photogrammetric techniques based on aerial photographs of the study reach that were taken on

February 19, 2006. Feature data, including buildings, vegetation, hydrography, and road surfaces, were added to the topographic data to finalize the base mapping in January 2008.

HEC-RAS Hydraulic Structures

Thirty-six bridges crossing the Lower Green River within the study reach were coded into the HECRAS model. Detailed information on some of the bridges was obtained from the effective FIS study model. However, most of the bridges were either newer than the effective study or simply not included in the effective model. Data for these structures were obtained from as-built drawings of the bridges and limited field surveys by nhc in December 2006. All bridge data were field verified to the extent possible by nhc.

HEC-RAS Starting Water Surface Elevation

The hydraulic model extends to the downstream study limit at RM 3.85. For calibration runs the starting water surface at the downstream end of the model was set equal to the observed tidal water surface elevation at the NOAA Seattle Station (No. 9447130) coincident with the peak of the simulated flood. For the flood profile simulations, the starting water surface elevation was set equal to the mean high water level at the Seattle station. It should be noted that the starting water surface used in the model has very little effect on simulated water surface elevations upstream of about RM 12.0 (e.g. lowering the starting water surface by 9 feet drops the BFE at RM 12 by about 0.08 feet).

HEC-RAS Model Calibration

Initial channel and overbank roughness factors (Manning's "n" values) used in the hydraulic computations were taken from the effective FIS model. These values were then adjusted by calibration to four observed flood events and one low flow event. High water mark data were collected by nhc and MGS for the high flow events of January 7, 2006, November 11, 2006, and March 25, 2007. High water mark data were also obtained from King County and other sources for the February 8-9, 1996 flood event.

Water level observations are available for 22 locations in the Lower Green Study reach during the January 2006 flood event. Corresponding high water mark data are also available for five of these locations. The January event had an estimated peak discharge of 11,200 cfs at the USGS gage near Auburn. Nine high water marks are available for the November 2006 flood and 11 were recovered for the March 2007 flood event. The November event had an estimated peak discharge at the Auburn gage of 12,200 cfs while the estimated peak discharge for the March 2007 event was 8,500 cfs. For the February 1996 event, nhc had HWM information at two locations, the Black River Pump Station Discharge Channel and the USGS gage at Auburn. In addition, approximate HWM information was extracted from oblique aerial photographs taken by King County near, or slightly after, the peak of this event. In all, 20 HWMs were identified on the photographs with elevations estimated from the topographic data created for this study by 3DI - West.

In addition to calibration to the high flow events, the model was also calibrated to the water surface elevation observed at the time of the aerial flight on February 19, 2006. The flow was about 1,100 cfs at the Auburn gage.

The final calibrated "n" values for the Lower Green River channel ranged from 0.026 at the upstream end of the study reach to 0.047 in the reach near the Horseshoe Acres pump station in Kent (RM 24.26 - 28.21). In general, Manning's "n" values downstream of RM 29 ranged from 0.039 to 0.047.

These in-channel roughness values account for both the heavily vegetated channel banks as well as the main channel bed. An attempt was made to define separate "n" values for the banks and bed but the composite "n" value computation that is used in HEC-RAS whenever multiple "n" values are defined between the bank stations results in unreasonable values for the Lower Green River. Thus, manually estimated composite "n" values were used for the channel.

Overbank "n" values used in the HEC-RAS model range from:

- 0.02 for roadways and other paved surfaces
- 0.04 to 0.055 for turf grass or pastured agricultural areas
- 0.07 to 0.09 for areas of thick brush or other dense vegetation.

The model was calibrated to reproduce observed stages for each of the calibration events as closely as possible, while bearing in mind that the most important event for Flood Insurance Study purposes is the 1-percent-annual-chance event and thus calibration to the highest observed discharges is more valuable than to the lower flow events. High water marks were generally matched to within 0.5 feet although in a few instances the difference between simulated and observed water levels was as great as 1 foot.

FLO-2D Model Development

Simulation of the complex floodplain hydraulics under the levee failure scenarios is not feasible with the HEC- RAS 1-Dimensional model. Therefore, a FLO - 2D model was developed for the floodplain and channel between RM 29.7 (near the Auburn Golf Course) and RM 10.6 (just downstream of the Black River Pump Station). The extents of the FLO-2D model are, compiled on February 12, 2008, was used in this study.

A grid size of 180 feet was selected, resulting in a model domain of approximately 21,000 cells. Grid cell elevations were calculated by averaging elevations of a l0-foot resolution DEM produced from the 3DI- West TIN. Prior to averaging, areas of fill such as roadway embankments that would be modeled as levees were removed from the DEM to avoid biasing elevation values upwards. River bank grid cell elevations were set equal to the natural bank elevations to ensure accurate bank overflow rates in the levee failure simulations. Mill Creek, Mullen Slough and lower Springbrook Creek were simulated by setting the elevations of individual grid cells to approximate channel thalweg elevations, and adding width reduction factors as necessary to match actual channel widths.

Area reduction factors (ARFs) are used in FLO-2D to handle reductions in flood storage within cells (e.g. due to buildings). These ARFs were calculated on an individual grid cell basis by setting the ARF value to the percent of the cell covered by structures as seen in the feature data coverage developed for the topographic base mapping. Width reduction factors (WRFs) are used to account for conveyance loss caused by structures obstructing the flow paths FLO-2D computes in each of the eight primary compass directions (i.e. N, NE, E, SE, etc.). WRFs were calculated by subdividing each model cell into a 3x3 sub-grid (9 sub-grid cells of 60 feet by 60 feet) and determining the percent of the area covered by structures within each sub-grid cell. The WRF for each flow path direction was then set to the larger of the covered area percentages for the pair of sub-grid cells that defined that particular flow path.

FLO-2D channel cross sections were generated by first using the interpolation tool in HEC-RAS to produce a cross section corresponding to each grid cell along the channel alignment. These cross sections were then clipped vertically at the natural bank elevations, which were determined on a cell by cell basis by taking the average elevation along a profile line delineated in GIS just outside the outer toe of the levee or on the top of the natural bank. Levees along the Green River, railroads, and roadways on fill were coded as levees in the model. Levee crest elevations were taken from the topographic mapping.

The FLO-2D model's downstream boundary condition was set as a rating curve generated by running a series of flows through the HEC-RAS model and fitting a best-fit power function though the resultant stage-discharge points.

Three hydraulic structures were incorporated in the model using rating tables: the Mill Creek (Auburn) culvert at SR-167, the lowest Mill Creek (Kent) bridge at the BNSF Railroad and the Black River Pump Station. A rating table for the SR-167 culvert was generated from an existing nhc FEQ model of Mill Creek. Preliminary modeling of floodway alternatives revealed that the Mill Creek (Kent) bridge opening under the railway tracks just upstream of its confluence with Springbrook Creek is a critical control on water levels over a large area upstream. Therefore, this opening was simulated using a rating table generated using the Springbrook Creek HEC-RAS model, which has far greater detail on the multiple bridges, culverts and channel cross sections that control the rating at this location. As discussed above, the Black River Pump Station was simulated using the nominal pump rating curve, with the contribution of local interior drainage flows (the 9-day, 1-percent-annual-chance flow) subtracted from the capacity. It should be noted that most of the bridges over the Green River within the study reach have clear spans, do not experience pressure flow, have their abutments behind the river levees, and present little or no constriction to flow. The water surface profiles in the HEC-RAS model and observed high water marks confirm this, showing no significant backwater effects at bridges. Therefore the bridges were not explicitly included in the FLO-2D model. The reasoning behind this decision is discussed in detail below.

In the HEC-RAS model, the bridges were all modeled in the “low flow-energy” mode which converts bridges to standard step-backwater cross sections. In the FLO-2D model, the upstream face cross section of each bridge in the HEC-RAS model was extracted for use as the cross section for the grid cell containing that bridge. The result is that the two models use similar approaches for bridges using only cross sectional data tables in the solution of the hydraulic equations. The only difference between the models is in the numbers of cross sections at each bridge (4 for HEC-RAS versus 1 for FLO-2D), and in the basic solution algorithm, with HEC-RAS using steady state step backwater methods and FLO-2D using an explicit unsteady flow solver. Special rating tables are not used for bridges in either model.

Discussions with FLO-2D developer Dr. Jim O'Brien and USACE staff on another NHC project (Skagit River) revealed that inclusion of bridge rating tables in FLO-2D was a common problem, and the tables typically needed modification from the HEC-RAS outputs to calibrate correctly, even where the bridge was not hydraulically significant. Given the number of bridges in the Green River system, that the bridges are hydraulically insignificant, that adding rating tables for every bridge promised to add significant stability and calibration complexities to the model, and that the FLO-2D and HEC-RAS models treated bridges in a similar way, it was decided that the selected modeling approach for the Green River bridges was appropriate.

FLO-2D Model Calibration

The FLO-2D Model was calibrated to the February 8-9, 1996 flood event. High water marks from the November 2006 event (with a peak flow at the Auburn gage of only 200 cfs less than the Feb 1996 event) showed a very similar profile above RM 18 and were also used to guide and validate the model calibration. Downstream of RM 18, the observed high water mark profiles for the 2 events diverge, and greater weight was given in the calibration to the higher February 1996 HWMs. The average absolute error of the calibrated profile to the 18 high water marks from the February 1996 event was 0.37 ft, and the largest error was -0.86 ft. Calibrated in-channel Manning's "n" values ranged from 0.035 to 0.045, except at the Auburn Mill Creek and Mullen Slough confluences, where single cell values of 0.050 were used to dampen transient surges in the solution.

Calibration of the model in the floodplain areas is not possible because the areas protected by levees have not experienced flooding during any of the recent high flow events. Therefore, Manning's "n" values for the floodplain areas were set based on engineering judgment and past experience. The floodplain was delineated into zones of similar roughness based on vegetative and land-use coverage and the FLO-2D model grid cells were attributed accordingly. Floodplain roughness values ranged from 0.04 for open pasture, golf courses and parks, 0.075 for large warehouse industrial areas, 0.08 for dense residential areas, up to a maximum of 0.09 for thickly forested and brushy wetlands.

Approximately 30 individual levee elements (facilities) were identified along the Lower Green River study area, primarily from the King County facility inventory. Some of these levee elements did not fully block the flow of water from the riverward side to the landward side of the levee, either because they had openings, e.g. culverts through the levee prism, or they were not high enough to contain the 1-percent-annual-chance event. Mapping of the area behind these levees, therefore, was done by transferring the in-channel BFE to the area behind the levee. The remaining levee elements were then grouped into "failure reaches" based on the

landward area that would be inundated if the leveed reach was failed.

In other words, all levees and appurtenant structures that protect the same land area are combined into a single levee failure reach. Combining the levee elements resulted in a total of five levee failure scenarios; Reddington, Reddington plus Mill Creek/Mullen Slough, Horseshoe Bend, Midway and Johnson Creek, and the East Valley. The five levee failure scenarios were analyzed individually using FLO-2D. In addition, the "all levees intact" condition, and the "fail all levees" condition were simulated. The floodplain map described below is a composite of the highest simulated water surface elevations at each grid cell from among these simulations.

An additional consideration for the levee failure analyses was the model's representation of the Black River Pump Station (BRPS). The currently installed nominal pump capacity of the BRPS is approximately 2,945 cfs (King County, 2007). The flow from the Green River into the East Valley area under the "East Valley" and "Fail-all" levee failure scenarios is about 2,200 cfs at its equilibrium condition. This flow would ultimately need to be pumped back into the Green River via the BRPS. The BRPS must also handle interior drainage for the East Side Green River watershed which is estimated at approximately 350 cfs for the 1-percent-annual-chance, 9-day condition. Based on these considerations, the installed BRPS pump capacity is adequate to handle the simulated pump station inflows without requiring storage of Green River floodwaters at the BRPS. To confirm the pumping capacity of the BRPS and verify routine operational needs, King County conducted a rigorous inspection and maintenance project of the BRPS during 2007 and 2008. During the inspection, all elements of the BRPS system were visually and mechanically tested. In the summer of 2008 as part of necessary maintenance, accumulated sediment was removed from the pump bays (i.e., chambers) of each pump. Subsequent to the sediment removal and with adequate forebay water levels, each pump was operated to ensure its functionality. Historically the BRPS has handled local stormwater runoff using only five of the eight pumps available at the facility. However, as described above, King County has verified that the full installed capacity is operational. In addition, under the newly formed countywide Flood Control Zone District, the County has the financial capability to provide the staff resources for operations and inspections, routine maintenance, and repairs as necessary to ensure the BRPS can continue to operate up to its design capacity. Therefore, under the scenario of an assumed levee failure of the Green River, the County and four valley cities mutually supported using the full pumping capacity (2,945 cfs) in the flood study modeling. The design pumping capacity would be adequate to handle both the interior runoff and the simulated Green River overflows. The FLO-2D model was therefore configured to use the BRPS design capacity (reduced by 350 cfs to account for interior runoff).

For the purposes of this flood study, only the certified Tukwila 205 Levee is recognized in the modeling and mapping. All other levees in the Lower Green River are not certified and thus are not recognized as providing flood protection for purposes of this mapping study. The FLO-2D model was used to analyze levee failure scenarios for all levees in the middle reach (between RM 10 and RM 30). The floodplain extents and depths of inundation were simulated for the five levee failure scenarios as described above. A composite floodplain map for the overbank areas was developed showing the worst case flooding from these simulations.

The final floodway configuration includes an area along the main channel corresponding to the HECRAS floodway, the Mill Creek off-channel floodway and the split flow floodway path on the right bank near RM 15. Review of the proposed floodway shows that the Mill Creek (Auburn) off-channel floodway was reduced slightly because simulated base flood elevations in this area are slightly lower than those previously mapped. The Springbrook Creek floodway was retained as delineated on the Preliminary DFIRM, with a minor expansion near the SW 41st Street crossing, and used to pass flows which exit the Lower Green River near RM 15 and return to the river via the Black River Pump Station. Validation of the adequacy of the Springbrook Creek floodway was demonstrated by routing flows calculated with the FLO-2D model down Springbrook Creek floodway and verifying that the water surface elevation did not exceed the FLO-2D "fail all" water surface by more than 1 foot at any location.

No profiles were generated for overbank areas modeled using the FLO-2D model. However, base flood elevations for these areas were established by manually fitting BFE contours at 1-foot increments to the grid of the maximum simulated water surfaces generated from the FLO- 2D results.

Green River (Middle Green River) – A steady-state HEC-RAS computer model was created to simulate the hydraulic characteristics of the study reach. The model was used to compute water surface profiles corresponding to the 10-, 2-, 1-, and 0.2-percent-annual-chance discharge, flood inundation limits for the 1-percent-annual-chance (i.e. base flood) and 0.2-percent-annual-chance events, and the floodway boundary for the 1-percent-annual-chance flood.

The following sections provide detailed descriptions of the development and application of the HEC-RAS model for this study.

Channel and Floodplain Topography

Seventy-three cross sections are used in the HEC-RAS model to represent the channel and floodplain geometry along the study reach. The in-channel portions of most of these cross sections were surveyed by Minister-Glaeser Surveying (MGS) in August and September 2006. Additional cross sections were interpolated from the survey and topographic data where needed. The in-channel cross section surveys typically only included the stream channel from bank to bank. The floodplain was not surveyed; therefore, the overbank portion of each cross section was added using the digital topographic data developed for this study by 3Di-West. The topographic data was created using a photogrammetric techniques based on aerial photographs of the study reach that were taken on February 19, 2006.

Hydraulic Structures

Six bridges cross the Middle Green River within the study reach. These are located at:

- State Highway 18 (two spans) at RM 33.27,
- Burlington Northern Railroad Crossing at RM 33.32,
- SE Auburn - Black Diamond Road at RM 33.38,
- SE Green Valley Road at RM 34.62,

- 218th Avenue SE (Whitney Bridge) at RM 41.20, and
- SE Flaming Geyser Road at RM 42.54

Detailed information on the bridges was obtained from an earlier floodplain mapping study performed for King County by Harper Houf Righellis, Inc. (HHR, 1995). These data were field verified to the extent possible by nhc and additional data were collected as necessary in March 2007.

Starting Water Surface Elevation

The hydraulic model was extended downstream to include the Lower Green River Study reach to provide the downstream starting water surface elevation. This approach ensures that the hydraulic effects of the large logjam near RM 32.3 were reflected in the modeling. The cross section and topographic data collected in 2006, provided the most reasonable and representative approach for establishing starting downstream water surface elevations. It is not anticipated that future refinements in the downstream HEC-RAS model will have any significant effect on water surface elevations at the SR-18 Bridge.

Model Calibration

Initial channel and overbank roughness factors (Manning's "n" values) used in the hydraulic computations were selected using engineering judgment and were based on field observations, orthophotos, published data, and values used in the previous flood studies. These values were then adjusted by calibration to three observed flood events and one low flow event. High water mark data were collected by nhc and MGS for the high flow events of January 7, 2006, November 11, 2006, and March 25, 2007.

High water mark data are available for seven locations in the study reach for the January 2006 flood event which had estimated peak discharges ranging from 9,480 cfs at the upstream end of the study to 11,070 cfs at the downstream end. Two high water marks were available for each of the November 2006 and March 2007 flood events. The November event had estimated peak discharges ranging from 10,250 to 12,170 cfs while the estimated peak discharges for the March 2007 event ranged from 8,070 to 8,730 cfs. In addition to calibration to these high flow events, the model was also calibrated to the water surface elevation observed at the time of the aerial flight on February 19, 2006. The flow at this time ranged from 730 cfs at the upstream end of the study reach to 1,100 cfs at the downstream end.

The final calibrated "n" values for the main channel range from 0.026 at the downstream end of the study reach to 0.045 in the reach through Flaming Geyser State Park. Overbank "n" values range from:

- 0.02 for roadways and other paved surfaces
- 0.04 to 0.055 for turf grass or pastured agricultural areas
- 0.07 to 0.09 for areas of thick brush or other dense vegetation.

The model was calibrated to reproduce observed stages for all calibration events as closely as possible, while bearing in mind that the focus of the model is for the 1-percent-annual-chance event. High water marks were generally matched to within 0.25 feet, although in some cases the difference between simulated and observed water levels was as great as 0.5 feet.

Floodplain Discussion

Within the limits of this floodplain mapping study, the Middle Green River can be divided into two general reaches. The upstream reach, from River Mile 44.3 to about RM 39 is relatively steep with a single thread and relatively straight channel and a floodplain that is generally confined to the main channel at most locations. Downstream of RM 39 to the downstream study limit at RM 33.25, the channel has a lower gradient, includes several large meander bends, and the floodplain is much wider than the channel and often covers the valley from side wall to side wall. The width of the floodplain in the upper reach is generally 200 feet to 500 feet wide. The floodplain in the lower reach ranges up to 2,500 feet wide or wider. Overbank inundations in the lower reach range from a few feet, near the transition between the upper and lower reach, to 6 feet or more near the downstream end of the study near RM 34.5

Table 4, "Manning's "n" Values," Table 6 (located in Volume 2 of the FIS), "Floodway Data," and the Flood Profiles were revised to reflect the results of the study.

3.2.9 Revision 9

Sammamish River and White River - A steady-state HEC-RAS, Version 4.0, computer model (Reference 188) was developed to stimulate the hydraulic characteristics of the White River. The HEC-RAS model was used to compute water surface profiles corresponding to the estimated 10-, 2-, 1-, and 0.2-percent-annual-chance floods, map flood inundation limits for the 1-percent-annual-chance (i.e. based flood) and 0.2-percent-annual-chance events, and define the floodway boundary for the 1-percent-annual-chance flood.

Channel and Floodplain Topography

One hundred and seventeen surveyed cross-sections were used in the Sammamish River model geometry. The in-channel portions of these cross-sections were surveyed by Pacific Geomatic Services, Inc. (PGS) in April 2009. The bathymetric surveys for the cross-sections included the river channel from the bottom of the channel to just above the water surface elevation at the time of the surveys; the overbank portion of each cross-section was obtained from the digital topographic data developed for this study by 3Di-West. This topographic data was created using photogrammetric techniques based on aerial photographs taken in March 2009. Two-foot contour topographic maps at a map scale of 1 inch equals 200 feet were developed for this study.

Thirty-eight cross sections are used in the White River model. The in-channel portions of most of these cross-sections were surveyed by Minister-Glaeser Surveying (MGS) in April and May 2007. Additional cross-sections were interpolated from the survey and topographic data where needed. The surveys for the most of the cross-sections only included the river channel from bank to bank. Where the floodplain was not surveyed, the overbank portion of each cross-section was added using the digital topographic data developed for this study by 3Di-West. The topographic data was created using photogrammetric techniques based on aerial photographs that were taken on March 29, 2007. Two-foot contour topographic maps at a map scale of 1 inch equals 200 feet were developed.

The cross-section information sources included "Field Survey" indicating the channel was directly surveyed in the field by MGS or NHC, "Topographic Map" meaning the channel or floodplain geometry was extracted from the topographic contours and "Interpolated" signifying the cross-section was interpolated from the upstream and downstream sections in the HEC-RAS model.

Hydraulic Structures

Hydraulic structures in the Sammamish River include numerous bridges with piers and/or abutments located throughout the river and the weir at RM 13.3. Detailed information on these structures was obtained from various state and local agencies, including Washington Department of Transportation, the City of Bothell, and the City of Redmond. These data were field verified to the extent possible by PGS in March 2009 and NHC in October 2009.

Hydraulic structures include the State Highway 410 Bridge at RM 22.45 and the White River Hydro Project diversion weir at RM 23.6. Detailed information on these structures was obtained from various state and local agencies, including Washington Department of Transportation, King County, the USACE, and Puget Sound Energy. These data were field verified to the extent possible by NHC in March 2008.

Starting Water Surface Elevations

Lake Washington stage, recorded at the Hiram M. Chittenden Locks, was used as the downstream boundary condition for the 60-year unsteady flow simulation period. Observed data were available only back to 1991 so a constant stage of 16.5 feet (NAVD 88) was used prior to that date for the unsteady HEC-RAS simulations. The stage of 16.5 feet (NAVD 88) corresponds with winter lake stages when peak flows typically occur on the Sammamish River. The observed stage at the Locks was converted to NAVD 88 using the following: NAVD 88 = USACE Lake Washington Datum – 6.82 feet (conversion to NGVD 29) + 3.6 feet (conversion to NAVD 88) +0.25 feet (to account for the average increase in lake stage between the Locks and Kenmore, (Reference 187).

The steady state modeling required a constant tailwater elevation. For the steady state HEC-RAS modeling, a tailwater condition of 18.5 feet (NAVD 88) was used corresponding to the maximum normal operating level of the Lake (the annual variation in lake stage is between 16.5 and 18.5 feet (NAVD 88). By using the annual maximum instead of the typical winter stage used in the unsteady analysis (16.5 feet NAVD 88), the resulting 1-percent-annual-chance event hydraulic and mapping results in a stage near the typical summer time river stage (instead of below it) near the downstream end of river. The significance of choosing the higher, but summer time, water level in Lake Washington was investigated by a sensitivity analysis. The sensitivity analysis varies the downstream boundary condition over the range of observed flows 16.5 to 18.5 feet (NAVD 88) and showed that there is relatively small change in upstream river stage. For the 1-percent-annual-chance event, there is less than 0.7 feet difference at the 68th Avenue Northeast Bridge (1,900 feet upstream) and less than 0.2

feet at the 96th Avenue Northeast Bridge when specifying the two different boundary conditions.

The starting water surface elevation for the HEC-RAS hydraulic model was obtained using a normal depth approximation, with a slope set to a value of 0.0072 ft.

Model Calibration

Initial channel and overbank roughness factors (Manning's "n" values) used in the hydraulic model were selected based on field observations, orthophotos, published data, values used in the previous FISs, and engineering judgment. The model was calibrated to high water marks along the length of the river and stage and flow hydrographs, when available, at the Willows Run gage and at the weir for three observed flood events (Reference 189). The Willows Run gage (King County gage 51t/USGS gage 12125200) has observed data from 1965 to 2009, and the weir (King County gage 51m) from 2001 to 2009. High water marks were generally matched to within 0.25 feet, but all values were within 0.5 feet. Manning's "n" values were varied for these events to reflect the increase in bank vegetation growth over time. The January 2009 calibration, representing current conditions, was used for the 60-year simulation and final analysis of the 10-, 2-, 1-, and 0.2-percent-annual-chance events and floodway.

The final calibrated "n" values for the channel ranged from 0.035 to 0.06 for the January 2009 event. The final calibrated "n" value for the overbanks ranged from 0.03 to 0.15 as listed below

High water mark data were collected by NHC and MGS for the high flow event of January 11-12, 2006. High water marks were available at six locations for the January 2006 event. The corresponding flows estimated for these marks ranged from 6,090 cfs at the upstream end on January 12th, to an event peak discharge of 10,750 cfs at the downstream end. High water mark data were available for the flood event in November 2006 at the two USGS gaging stations in the reach (No. 12098500 and No. 12099200). The USGS estimated peak discharge at the downstream gage, above the confluence of Boise Creek, was 14,700 cfs. A single high water mark was available for the December 2007 event. The discharge corresponding to that mark was estimated at 6,830 cfs.

With and Without Levees Modeling

FEMA requires a “without levee” analysis be conducted for the 1- and 0.2-percent-annual-chance event flows when non-certified levees are present. This is done in addition to a simulation that allows the “levees” to hold back water. The higher water surface elevations between the “with levee” and “without levee” simulations are then mapped. Berms along the Sammamish River banks, acting like “levees,” obstruct water from getting into the floodplain. These berms are not certified as levees. Therefore, to meet FEMA requirement, the hydraulic analysis for this study assumes that these berms do not hold back water. This affects the extents of inundated area for the 1-and 0.2-percent-annual-chance events. The floodway analysis (Section 4.2) assumes that the berms do not provide flood protection.

Floodplain Discussion

The Sammamish River can be divided into two general reaches of differing characteristics. The upstream reach, from Lake Sammamish down to approximately RM 6.2, has a wide valley relative to the lower section and a flat valley floor. Broad overbanks are exposed to potential inundation. Distances from valley wall to valley wall in some areas are upwards of 4,500 feet. Downstream of RM 6.2, the valley narrows considerably, with 1,000 feet or less between valley walls. The channel is also more sinuous in the downstream reach.

White River can be divided into 2 general reaches of differing characteristics. The upstream reach, from Mud Mountain Dam at RM 28.6 to about RM 27.0 is very steep and is confined within a canyon with steep sidewalls. The river has a single thread and the floodplain is generally limited to the area between the channel banks. The width of the floodplain in this upper reach is generally 100 feet to 250 feet. Downstream of RM 27 to the downstream study limit at RM 22.01, the channel has a slightly lower gradient and includes several large meander bends and side channels. The floodplain in the lower reach is often much wider than the channel and in some locations extends from valley wall to valley wall. The floodplain in the lower reach ranges from several hundred up to 1,600 feet wide. Overbank inundation in the lower reach of the studied area ranges from a few feet to 6 feet or more in the low lying side channels where flow splits occur.

The Manning’s “n” values for all detailed studied streams are presented in Table 4.

Table 4. Manning's "n" Values

Stream	Channel "n" Range	Overbank "n" Range
Bear Creek	0.040-0.100	0.060-0.300
Big Soos Creek	0.024-0.090	0.040-0.150
Black River	0.011-0.050	0.050-0.150
Cedar River	0.02 - 0.045	0.03 - 0.15
Coal Creek	0.035-0.042	0.055-0.075
Des Moines Creek	0.030-0.040	0.050-0.100
East Branch of West Tributary Kelsey Creek	0.035-0.042	0.055-0.075
East Fork Issaquah Creek	0.035-0.060	0.050-0.250
Evans Creek	0.039-0.063	0.056-0.135
Forbes Creek	0.045	0.050
Gardiner Creek	0.070-0.080	0.070-0.200
Gilman Boulevard Overflow	0.040-0.045	0.030-0.045
Green River	0.020-0.055	0.060-0.300
Holder Creek	0.030-0.055	0.020-0.120
Issaquah Creek	0.030-0.088	0.035-0.300
Kelsey Creek	0.035-0.042	0.055-0.075
Little Bear Creek	0.012-0.080	0.016-0.150
Longfellow Creek	0.025-0.065	0.065-0.070
Lyon Creek	0.025	0.050
Maloney Creek	0.037-0.055	0.050-0.100
May Creek	0.030-0.090	0.055-0.150
May Creek Tributary	0.040	0.070
McAleer Creek	0.025-0.050	0.013-0.080
Mercer Creek	0.035-0.042	0.055-0.075
Meydenbauer Creek	0.035-0.042	0.055-0.075
Middle Fork Snoqualmie River Overflow Channels	0.040-0.045	0.075
Mill Creek (Auburn)	0.012-0.090	0.045-0.095
Mill Creek (Kent)	0.012-0.041	0.050-0.120
Miller Creek	0.040-0.050	0.060-0.120
North Branch Mercer Creek	0.035-0.042	0.055-0.075
North Creek	0.030-0.055	0.050-0.100
North Fork Issaquah Creek	0.026-0.055	0.070-0.120
North Fork Meydenbauer Creek	0.035-0.042	0.055-0.075

Table 4. Manning's "n" Values *(Continued)*

Stream	Channel "n" Range	Overbank "n" Range
North Fork Thornton Creek	0.012-0.045	0.028-0.120
Patterson Creek	0.040-0.120	0.020-0.080
Raging River	0.035-0.080	0.050-0.090
Ribary Creek	0.045-0.048	0.050-0.120
Richards Creek	0.035-0.042	0.055-0.075
Richards Creek East Tributary	0.035-0.042	0.055-0.075
Richards Creek West Tributary	0.035-0.042	0.055-0.075
Right Channel Mercer Creek	0.035-0.042	0.055-0.075
Rolling Hills Creek	0.025-0.040	0.020-0.060
Sammamish River	0.026-0.057	0.027-0.042
Snoqualmie River (Mainstem)	0.030-0.055	0.020-0.150
Snoqualmie River (Middle and North Forks)	0.028-0.058	0.040-0.170
South Fork Skykomish River	0.038-0.048	0.080-0.120
South Fork Snoqualmie River	0.038-0.100	0.070-0.120
South Fork Thornton Creek	0.012-0.045	0.028-0.120
Springbrook Creek	0.050-0.070	0.030-0.040
Swamp Creek	0.045-0.085	0.050-0.120
Thornton Creek	0.012-0.045	0.028-0.120
Tibbetts Creek	0.027-0.055	0.080-0.130
Tolt River	0.042-0.055	0.070-0.100
Vasa Creek	0.035-0.042	0.055-0.075
Walker Creek	0.050	0.060-0.120
West Fork Issaquah Creek	0.024-0.050	0.035-0.120
West Tributary Kelsey Creek	0.035-0.042	0.055-0.075
White River	0.027-0.057	0.015-0.99
Yarrow Creek	0.045	0.150